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Report of SUBCOMMITTEE 7, ACI COMMITTEE 325, "Review of the Use of Continuous Reinforcement in Highway Pavements" will appear in December along with "Stress Distribution Affects Ultimate Tensile Strength," submitted by JAMES S. BLACKMAN, GERALD M. SMITH, and LYLE E. YOUNG. The latter study was conducted by the authors at the University of Nebraska.

●

Experience in design and manufacture of "Lightweight Concrete Deck for Tappan Zee Bridge Main Spans" will be related by W. G. MULLEN.

E. M. RENSAA discusses "Problems of Shear and Anchorage in Reinforced Concrete Beams" with the object of creating a better understanding of shearing stresses and related problems in reinforced concrete.

●

Coauthors D. WATSTEIN and R. G. MATHEY present "Strains in Beams Having Diagonal Cracks," based on research at the National Bureau of Standards.

K. C. RAY offers a set of " Influence Lines for Pressure Distribution Under a Finite Beam Resting on Elastic Foundation."

●

In "Application of the Specific Surface of Aggregates to Concrete Mix," B. G. SINGH suggests a proportioning method that makes use of this property as an index of grading; method is applicable to both continuous and gap-graded aggregates.

"Wear Resistant Concrete Construction," by SHU-T'IEN LI sets forth the requirements for producing concrete surfaces to withstand heavy traffic or severe treatment.

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Title No. 55-34

Proposed ACI Standard

Recommended Practice for

HOT WEATHER CONCRETING*

Reported by ACI Committee 605

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SYNOPSIS

This recommended practice provides information useful in minimizing detrimental effects of hot weather on concrete. Means are described for reducing concrete temperature by proper attention to ingredients; methods of production and delivery; and care in placement, protection, and curing. Information is given on the use of admixtures to reduce mixing water requirements and to retard setting. Emphasis is given to the importance of meticulous attention to the use of standard procedures in testing concrete made in hot weather.

INTRODUCTION

Hot weather presents special problems in manufacture, placement, and curing of portland cement concrete. High temperatures result in more rapid hydration of cement, greater mixing water demand, increased evaporation of mixing water, reduced strengths, and larger volume changes. Effects

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This report was submitted to letter ballot of the committee, which consists of 20 members, with 17 voting affirmatively and 3 not voting. It is released by the Standards Committee for publication and discussion with a view to its consideration for adoption as an Institute Standard at the 55th Annual Convention, Los Angeles, Calif., Feb. 23-26, 1959.

of hot weather on concrete, in the absence of special precautions, may be summarized as follows:

1. Setting is accelerated. High temperatures increase the rate of hardening of the concrete. The length of time within which the concrete is handled is more critical. Quick stiffening encourages undesirable retempering by addition of water, and it may also result in inadequate consolidation and cold joints.

2. Strength is reduced. High temperatures of plastic concrete appear to affect the gel formation during the process of hydration of the cement so as to result in lower strengths. High temperatures increase mixing water demand.

3. The tendency to crack, either before or after hardening, is increased. Plastic shrinkage cracks may form in the partially hardened concrete due to rapid evaporation of water. Cracks may be developed in hardened concrete either by increased drying shrinkage resulting from greater mixing water demand or by cooling of the concrete from its elevated initial temperature.

4. Need of adequate curing is emphasized. The time at which protective measures are applied is more critical, due to the difficulty of retaining moisture for hydration and maintaining reasonably uniform temperature conditions.

5. Control of air content in air-entrained concrete is more difficult. This adds to the difficulty of controlling slump. For a given amount of air-entraining agent, warm concrete will entrain less air than cool concrete.

The problems of concreting in hot weather have been increased in recent years by a number of factors, including more finely ground and more rapidly hardening cements, handling of larger batches of concrete, use of thinner sections, and increased speeds in all construction operations. For the most part, such special attention as has been given to minimizing the difficulties resulting from high temperatures has been limited to mass construction. The emphasis has been on hot concrete rather than hot weather. This recommended practice suggests procedures for reducing the hazards of hot weather in the more usual classes of construction such as buildings, bridges, highways, and miscellaneous structures.

The harmful effects of hot weather on concrete may be minimized by a number of practical procedures. The degree to which their application is justified depends on circumstances and should be determined in the light of informed engineering judgment.

One or more of the ingredients may be cooled to keep the temperature of the concrete from being excessive at time of placement. Cement content, because of heat of hydration, should be maintained at the minimum permitted by the work. Forms and reinforcing steel may be protected from direct rays of the sun and cooled by sprinkling. Suitable attention to scheduling of delivery and placement to avoid excessive mixing or delays contributing to stiffening will be beneficial. Concrete in place may be protected so as to minimize drying and absorption of heat. When excess water demand and too rapid stiffening are not prevented by reduced temperatures, protection, and other precautions, they may be compensated for by the use of suitable water-reducing retarders.

To accomplish the objectives outlined in the preceding, advance planning is necessary. Difficulties resulting from high temperatures, such as plastic shrinkage cracking, can only rarely be controlled by last minute improvisations. Damage from shrinkage and temperature cracks cannot be fully alleviated after cracks have formed.

TEMPERATURE OF CONCRETE AS PLACED

The optimum temperature of concrete at time of placement is lower than can generally be attained in hot weather.[1] A temperature of 60 F, or even lower, would be desirable. A maximum of 90 F should be considered a reasonable and practicable upper limit. However, it is cautioned that difficulty may be encountered with concrete at temperatures approaching 90 F and every effort should be made to maintain it at a lower temperature.

The most direct approach to keeping concrete temperature down is by controlling the temperature of its ingredients.[2,5,6,7] The contribution of each ingredient to the temperature of concrete is a function of the temperature, specific heat, and quantity used of that ingredient. A fairly accurate estimate of the temperature of fresh concrete can be calculated from established relationships which have been used for the most part in connection with winter concreting.* In warm weather, the actual temperature is likely to be slightly higher than the calculated temperature because the computations do not take into account heat of wetting and hydration of cement, nor that generated by the mechanical work of mixing. Calculations based on the formula in the footnote, using specific heat of dry material of 0.20, show that, for concrete containing 6 sacks per cu yd of cement with a water-cement ratio of 0.5 by weight, the temperature will be changed approximately 1 F by any one of the following changes in temperature of ingredients: cement 9 F; water 3.6 F; aggregate 1.6 F.

From the preceding it will be seen that aggregates and mixing water exert the most pronounced effect on temperature of concrete. Thus, in hot weather it is essential to use all available means for maintaining these materials at as low temperatures as practicable.

Aggregates

There are several procedures for lowering temperatures or at least preventing excessive heating of aggregates. Shading stockpiles from direct rays of the sun provides some benefit. Sprinkling stockpiles and keeping them moist results in cooling by evaporation, this procedure being especially effective

*$T = \dfrac{S\,(T_a W_a + T_c W_c) + T_f W_f + T_m W_m}{S\,(W_a + W_c) + W_f + W_m}$

where

W_a = weight of aggregate (surface dry), lb
T_a = temperature of aggregate, deg F
W_c = weight of cement, lb
T_c = temperature of cement, deg F
W_f = weight of free moisture in aggregate, lb
T_f = temperature of free moisture in aggregate, deg F
W_m = weight of mixing water, lb
T_m = temperature of mixing water, deg F
S = 0.20, assumed specific heat of dry materials
T = temperature of freshly mixed concrete, deg F

when relative humidity is low. When aggregates are stockpiled during hot weather, it is desirable for successive layers to be sprinkled as the pile is built up. If cold water is available, heavy spraying of coarse aggregate immediately before use will have a direct cooling action. On some mass concrete work,[3,6] aggregates have been cooled by inundating them in artifically cooled water or forcing refrigerated air through the bins.

Water

Pound for pound, the mixing water has the greatest effect of any of the ingredients on temperature of concrete, since it has a specific heat about 4½ to 5 times that of cement or aggregate. It should not be overlooked, however, that it is used in smaller quantities than the other ingredients. Even so, use of cool mixing water is effective. Every effort should be made to use water from a cool source and to keep it cool by protecting pipes and tanks.

Pipelines for conveying mixing water should be buried, insulated, shaded, or painted white. Water storage tanks should be treated in a similar manner. Tank cars or trucks used for transporting water should be insulated or painted white.

Under certain circumstances, reduction in water temperature may be most economically accomplished by mechanical refrigeration or mixing with crushed ice.[4,7] Use of ice as a part of the mixing water is highly effective in reducing concrete temperature since, on melting alone, it takes up heat at the rate of 144 Btu per lb. Thus, if ice were used as 50 percent of the mixing water in the 6-sack concrete considered above, the melting of the ice alone would lower the concrete temperature approximately 20 F and the resulting water at 32 F would have an additional cooling effect near 7 to 8 F. To take advantage of the heat of fusion, the ice must be incorporated directly into the concrete as part of the mixing water. Conditions must be such that the ice is completely melted by the time mixing is completed.

Cement

Hot cement[8] may result from grinding hot clinker or from heat generated mechanically during grinding. Since heat is lost slowly during storage, cement is often delivered at a relatively high temperature, particularly during periods of peak production or short supply when storage periods for both unground clinker and finished cement are likely to be short. Although, as already shown, changes in temperature of cement produce significantly less change in the temperature of fresh concrete than the other ingredients, it does exert an effect and, therefore, it is prudent to place a maximum limit on its temperature as it enters the concrete. Cement preferably should not be used at temperatures in excess of about 170 F. Other than its contribution to raising the temperature of concrete, high temperature of cement, as such, appears to have no detrimental effects.

USE OF ADMIXTURES TO RETARD HARDENING AND REDUCE MIXING WATER

Hydration of cement is accelerated by high temperatures. It seems probable that this acceleration is largely responsible for the increase in water requirement of concrete in hot weather. Certain water-reducing retarders may counteract the accelerating effect of high temperature and lessen the need for increase in mixing water. They are beneficial provided they do not interfere with strength development of the cement.

Certain retarders have been found to delay setting time of concrete, reduce mixing water requirement, and increase strength.[9] Advantages of such effects in hot weather are obvious. Consideration should be given to the use of suitable water-reducing retarders during hot weather, when temperatures are such as to increase mixing water demand or reduce slump significantly. Under some conditions these effects begin to become apparent when weather is such as to raise concrete temperature as placed consistently above 75 F. However, no admixture should be used without competent technical advice or, when practicable, advance testing with the cement and other concreting materials involved.[10]

Accelerators should be used during hot weather only under careful supervision. In conjunction with high temperatures and drying effects, they may cause the concrete to set too rapidly for satisfactory placement and finishing. Concrete made with high-early-strength cement will harden more rapidly than that made with other types and early initiation of protective measures is, therefore, all the more important.

PRODUCTION AND DELIVERY

Problems of production and delivery overlap those of materials. As already indicated, temperatures of aggregates, water, and cement should be maintained at the lowest practical levels so that the temperature of the concrete may be as low as is feasible.

The ready-mixed concrete operation presents some special problems. If cement is hot, care must be taken in batching to prevent its being dampened by a small amount of water before it is well dispersed with the other materials. Otherwise it may set quickly and cause cement balls to form.

During hot weather, care should be exercised to keep mixing at the minimum which will insure adequate quality. Elapsed time between mixing and placing should be minimized. When elapsed time and job conditions are such as to result in significant increases in mixing water demand or reduction in slump, truck mixing should be delayed until only time enough remains to accomplish it before the concrete is placed. Trucks should be dispatched so as to avoid delay and work should be organized to use the concrete promptly to prevent unnecessary additional mixing at the job site. Exposure of mixers to the hot sun while waiting to be unloaded should be minimized. Heat reflecting colors on mixer drums may be of assistance.

PLACEMENT AND PROTECTION

Difficulties from hot weather can be minimized by more rigid controls of placement and protection than are required at other times. In addition to controlling the temperature of the concrete as delivered to the work, much can be done with placement facilities and practices to keep temperatures low. Mixers, chutes, belts, pump lines, and forms should be shaded. Surfaces of such equipment as pump lines, if painted white, will absorb substantially less heat than when painted other colors. Aluminum painted surfaces, for example, may be 10 to 15 deg warmer in the hot sun than white, and black surfaces may be as much as 30 deg warmer. Covering surfaces with damp burlap, kept damp, provides low heat absorption and, also, cooling from evaporation.

Forms, reinforcing, and subgrade should be sprinkled with cool water just prior to placement of concrete. Wetting down the area around the work will cool the surrounding air and increase its humidity, thus reducing temperature rise and evaporation from the concrete. When temperature conditions are critical, improved results will be secured if concrete placement is restricted to the evening or night when temperatures are lower and evaporation is less.

Placement and finishing

Speed of placement and finishing helps to minimize hot weather difficulties. Delays contribute to slump loss and lead to use of additional mixing water to offset the reduction. Ample personnel should be available to handle and place concrete immediately on delivery. On flat work, all steps in finishing should be carried out promptly. Here again, adequate personnel, both as to number and skill, are important. Delays in finishing air-entrained concrete pavements in hot weather lead to formation of a rubbery surface which is impossible to finish without leaving ridges that impair the riding qualities of the pavement.

Other modifications in usual placement practices may be indicated. Concrete should be placed in layers thin enough and in areas small enough so that the time interval is reduced and vibration or other working of the concrete will insure complete union of adjacent portions. If cold joints tend to form, if surfaces set and dry too rapidly, or if plastic shrinkage cracks tend to appear, the concrete should be kept moist by means of fog sprays, wet burlap, cotton mats, or other means. Fog sprays, applied shortly after placement and before finishing, have been found to be particularly effective in preventing plastic shrinkage cracks when other means have failed. When these precautions are not sufficient, it may be desirable, as mentioned earlier, to supplement them by restricting work to the evening or night. If plastic shrinkage cracks do form, they can often be worked together during the finishing operation, although if only the surface is consolidated, relief is temporary and the cracks may open again later.[11,12] Suitable revibration,[13]

as late as the concrete will still become plastic under vibration, will do much to prevent plastic shrinkage cracks or to eliminate those which have already formed.

In summary, all placement procedures should be directed to keeping the concrete as cool as practical and assuring its setting and hardening under temperature conditions which are reasonably uniform and under moisture conditions which will minimize drying. Concrete, whether delivered by pump, truck mixer, or otherwise, should reach the forms at an acceptably low temperature and whatever is practicable should be done to minimize temperature increases during placing, finishing, and curing operations.

Protection and curing

Since hot weather leads to rapid drying of concrete, protection and curing are far more critical than during cool weather. Consequently, particular attention should be paid to having all facilities ready for prompt initiation of curing. Exposed surfaces should be carefully protected from drying. Wherever it is practicable, continuous water curing is to be preferred. Continuity is important because volume changes due to alternations of wetting and drying promote the development of pattern cracking. Need for adequate continuous curing is greatest during the first few hours after placement.

Reliance should not be placed on the protection afforded by forms for curing in hot weather. If practical, water should be applied to formed surfaces, while forms are still in place. Unformed surfaces should be kept moist by wet curing for at least 24 hr, and curing should be started as soon as the concrete has hardened sufficiently to withstand surface damage. Soil soaker hoses or spray nozzles should be provided to keep covering materials soaked. Steeply sloping and vertical formed surfaces should be kept completely and continuously moist prior to and during form removal by applying water to top surfaces so that it will pass down between the form and the concrete.

On exposed unformed concrete surfaces such as pavement slabs, wind is an important factor in the drying rate of concrete.[12] For example, other conditions being equal, a gentle wind of 10 mph will cause four or more times as much evaporation from a flat surface as still air. Wind breakers and covers have been used successfully not only to protect pavements or other flat surfaces from the sun but also to prevent wind from blowing over the surfaces prior to the initiation of curing.

If moist curing is not continued beyond 24 hr, the surface, while still damp, should be covered with a suitable heat-reflecting plastic membrane or sprayed with white-pigmented curing compound. The pigment not only reflects heat but simplifies checking the uniformity and extent of application of the compound. Need for adequate curing is greater during the first few days after placement.

On hardened concrete and on flat surfaces in particular, curing water should not be much cooler than the concrete because of temperature-change stresses which would be introduced, with resultant cracking. Where back-

filling is involved, it should be done as soon as safe and practicable. At the termination of curing with water, an effort should be made to reduce the rate of drying by avoiding air circulation. This can be accomplished by delay in removal of wet covers until they are dry. Closing tunnels, pipelines, tubes, and buildings reduces drying drafts.

CONCRETE TESTING

Accurate testing is always highly important. In hot weather, strict conformance to standard test methods becomes especially critical.

As discussed earlier, high temperature of concrete at early ages is detrimental to strength independently of side effects of increased water requirement.[1,14,18] In addition to the effects of heat, further damage may be done to test specimens in hot weather by loss of moisture by evaporation. Due to their small size in relation to most parts of the structure, test specimens are likely to reach higher temperatures and dry more rapidly than the concrete in place, with corresponding increased detrimental effects. For these reasons, extra care is needed in hot weather to maintain temperature and moisture conditions for strength test specimens as required in standard test methods.

Much that has been said concerning protection of concrete in the structure can be extended to apply to strength test specimens. Proper temperatures can be maintained by avoiding exposure to the sun and by utilizing the cooling effect of evaporating water, either from damp burlap or wet sand covering the specimens. Merely covering the test specimen with a lid or plate is not sufficient in hot weather to prevent loss of moisture. Damp sand, wet burlap, or fog sprays will insure retention of water by specimens.

Specimens used as the basis for acceptance of concrete as delivered to the structure must be transferred at the age of 1 day to a location (usually the laboratory) where they will receive continuous standard moist curing until test. This requirement, as in the case of provision for early protection, is of particular importance in hot weather because of the great damage to strength which results from high temperatures and drying.

Specimens cured on the job should never be substituted for laboratory-cured specimens as a check of the proportioning and mixing of the concrete. Results of tests of job-cured specimens, properly interpreted, may be used to yield information helpful in judging when to strip forms, put the structure in service, etc. The committee recognizes that better test procedures than are now available are needed to make an accurate determination of the quality of concrete as placed in the structure.

TEMPERATURE RECORDS

During hot weather, inspectors should record at frequent intervals air temperature, general weather condition (calm, windy, clear, cloudy), and relative humidity. The record should include checks on temperature of concrete as delivered and after placing in the forms. Data should be corre-

lated with the progress of work so that conditions surrounding the construction of any part of the structure can be ascertained. A copy of the weather data should be included in the permanent records of the job.

SUMMARY

Hot weather may adversely affect concrete by causing increased water demand, reduced strength, and excessive volume changes which result in cracking. The effects result from high concrete temperatures and rapid evaporation of water from the concrete. The difficulties can generally be minimized by

1. Lowering concrete temperatures by using cool ingredients, eliminating excessive mixing, preventing exposure of mixers and conveyances to direct sunlight, and using reflective paint on mixers and conveyances.

2. Preventing too rapid stiffening and keeping mixing water requirements low by eliminating delays or excessive mixing, and by judicious use of proved water-reducing retarders.

3. Proper protection and curing of both the structure and strength test specimens to prevent high temperatures or evaporation of water during hardening and critical strength development periods.

Specific methods for the accomplishment of these objectives have been outlined in this recommended practice. Additional information can be secured from the references in the appended list.

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Proposed Revision of ACI Standard 614-42

Recommended Practice for

Measuring, Mixing, and Placing Concrete*

Reported by ACI Committee 614

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SYNOPSIS

An outline of practices which have generally been found desirable for first class results in measuring, mixing, and placing concrete. Although many of these recommendations are applicable and should be used in connection with special types of concrete, i.e., lightweight, prepacked, etc., it is conventional concrete to which they specifically apply. Presents a comparatively high standard of practice rather than common practices; therefore recommendations are made on a "should" basis leaving to the user the responsibility of putting them on a "shall" basis in specifications for his work to the extent he considers worthwhile.

I. INTRODUCTION

This recommendation has been prepared as an outline of practices which have generally been found desirable for first class results in measuring and mixing ingredients for concrete and when placing it in the work. Although many of these recommendations are applicable and should be used in connection with special types of concrete such as lightweight, prepacked, porous,

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The report was unanimously approved by the committee listed above. It is released by the Standards Committee for publication and discussion with a view to its consideration for adoption as an Institute Standard to supersede "Recommended Practice for Measuring, Mixing, and Placing Concrete (ACI 614-42)" at the 55th Annual Convention, Los Angeles, Calif., Feb. 23-26, 1959.

vacuum, precast, prestressed, and poststressed concrete, shotcrete and mortar, it is not in the scope of these recommendations to include all practices which should be followed for such special types of concrete.

In making these recommendations, the committee has been guided by the belief that progress in improvement of concrete construction practices will be better served by endorsing a comparatively high standard of practice rather than "common practices." Because some may consider something less than these relatively high standards sufficient for their purpose, this recommended practice is presented on a "should" basis. It will be the responsibility of the user to put it on a "shall" basis in specifications for his work, to the extent he considers worthwhile, and to see that specified results are obtained through good inspection. ACI Committee 614 has no evidence that operations resulting in good concrete cannot in general be performed as economically as those often productive of poor concrete. Nor will the methods here recommended result in unjustifiably higher costs than methods productive of inferior concrete. Many of the practices recommended are included primarily to improve concrete uniformity. Where the methods described have been carried out properly, the effort and investment have usually been rewarded by concrete of higher quality, a smoother operation, and higher production rates, all factors which tend to offset any additional cost.

It has been assumed that anyone giving serious consideration to employing these recommendations will have some knowledge of the ordinary practices and preparations required for good concrete work. For this reason, many routine instructions for measuring, mixing, and placing concrete are omitted, particularly since they have been well covered in the *ACI Manual of Concrete Inspection*.[1] Since the main objective of this recommendation is maximum uniformity, homogeneity, and quality of concrete in place, special consideration is given to practices which have been found to contribute most to this end. To portray more clearly certain of the principles involved in these practices, five figures illustrating examples of good and bad practice are included as a part of these recommendations.

For further discussion of measurement, mixing, and placing, the reader is referred to the bibliography. Attention is called particularly to References 1, 2, 3, and 4. ACI Committee 614 is, in general, in accord with the recommendations of the references, although a few exceptions in detail or in emphasis may be noted.

II. AGGREGATES

1. Fine and coarse aggregate must be uniform in grading and moisture content on arrival at batch weighing equipment if uniform concrete is to be attained. Unless such aggregate is assured, *as batched*, through appropriate specifications and effective inspection of selection, preparation, and handling of aggregates, production of uniform concrete is extremely difficult, and to a large degree unlikely, despite a high order of accuracy in measuring and superior performance in mixing and placing. Consequently, the committee

considers this brief statement on aggregate requirements an essential part of these recommendations.

2. Segregation in coarse aggregate is reduced to the practical minimum by separating the material into several size fractions and batching these fractions separately. As the range of sizes in each fraction is decreased and the number of size separations is increased, segregation is further reduced in each size. It is not necessary that the ratio of maximum to minimum size for each size separation of coarse aggregate be the same for all sizes. However, worthwhile control of segregation in all sizes, and control of the amount of pea gravel in the mix, are obtained when this ratio is held to not more than 3 for sizes smaller than 1 in. and to 2 for larger sizes. Examples would be

No. 4—½ in., ½-1 in., 1-2 in., 2-4 in., or
No. 4—½ in., ½-1¼ in., 1¼-2½ in., 2½-5 in., or
No. 4—⅜ in., ⅜-¾ in. (or ¼-¾), ¾-1½ in., 1½-3 in., 3-6 in.

3. (*a*) It is essential for effective control that handling operations (Fig. 1) be such that variations in undersize material outside each designated size, be held to a practical minimum. Undersize smaller than about 4/5 to 5/6 the designated minimum size of an aggregate is significant in distributing concrete uniformity and slump control,[5] particularly when the designated minimum size is smaller than 1 in. Moreover, it is the varying content of this "significant" undersize in aggregate *as batched*, that it is important to control, if uniform and efficient concrete mixes are to be obtained. Due to segregation, breakage, and contamination during handling, particularly if aggregates go through cars or stockpiles or both, it is practically impossible to control undersize uniformly within close limits when such aggregate is batched.

(*b*) For reasons cited in Section II(3)a, unless coarse aggregate is placed in batch plant bins directly from screening operations, finish screening at the batching plant is recommended as the method most likely to provide aggregate with undersize in excess of close limits removed at the last possible point before use (Reference 4, p. 177 and Fig. 76). Some prominent agencies have required finish screening for many years on projects ranging in size from a few thousand to several million cubic yards; some contractors have chosen to finish screen when it was not specified; in each case results were beneficial.

(*c*) With properly operated vibrating finish screens, it is quite practical to keep significant undersize smaller than 1 in., within 2 percent in batched aggregate. Due to tendencies to overfeed screens, horizontally operating vibrating screens have been found the most consistent and reliable for finish screening. Three percent is a practical limit for significant undersize smaller than 1 in. on small jobs where only a stationary sloping finish screen with slotted openings can be used economically. Generally the screens operate most effectively when mounted over the batch plant bins. Where an existing plant is not structurally suited to this arrangement, the finish screening equipment may be installed adjacent to the batching plant, either at ground

INCORRECT METHODS OF STOCKPILING AGGREGATES CAUSE SEGREGATION AND BREAKAGE

PREFERABLE

Crane or other means of placing material in pile in units not larger than a truck load which remain where placed and do not run down slopes.

OBJECTIONABLE

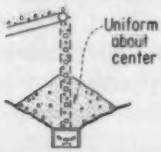
Methods which permit the aggregate to roll down the slope as it is added to the pile, or permit hauling equipment to operate over the same level repeatedly.

LIMITED ACCEPTABILITY—GENERALLY OBJECTIONABLE

Pile built radially in horizontal layers by bulldozer working from materials as dropped from conveyor belt. A rock ladder may be needed in this setup.

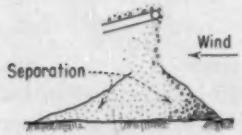
Bulldozer stacking progressive layers on slope not flatter than 3:1. Unless materials strongly resist breakage, these methods are also objectionable.

STOCKPILING OF COARSE AGGREGATE WHEN PERMITTED
(STOCKPILED AGGREGATE SHOULD BE FINISH SCREENED AT BATCH PLANT; WHEN THIS IS DONE NO RESTRICTIONS ON STOCKPILING ARE REQUIRED)



CORRECT

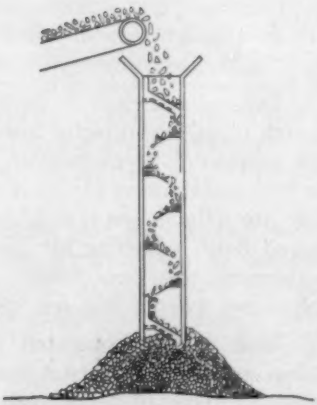
Chimney surrounding material falling from end of conveyor belt to prevent wind from separating fine and coarse materials. Openings provided as required to discharge materials at various elevations on the pile.



INCORRECT

Free fall of material from high end of stacker permitting wind to separate fine from coarse material.

UNFINISHED OR FINE AGGREGATE STORAGE (DRY MATERIALS)



When stockpiling large sized aggregates from elevated conveyors, breakage is minimized by use of a rock ladder.

FINISHED AGGREGATE STORAGE

Fig. 1—Correct and incorrect handling of aggregates

level or above the bins. In any case, the screened material should be conveyed directly to the proper bin.

(*d*) It is emphasized that requirement of these finish screening methods provides good assurance *in advance* that it will be possible to control objectionable undersize to 2 or 3 percent, generally, regardless of what aggregate situation may develop. But the main thing is the result, and if it *can be assured* by other means, there is no objection to them. Even with finish screening at the batching plant, it may be worthwhile in some cases to follow good practice in handling and stockpiling coarse aggregate.

(*e*) Where the batching demand for various sizes of coarse aggregate is fairly certain or regular, gradings are not disrupted seriously by reclaimed undersize from larger aggregate, if, with sufficient screen capacity, some of all sizes are finish screened together in varying proportions as necessary to keep all sizes at proper levels in the bins, or short runs are made of each size in sequence. In this operation, undersize in the sand range should be wasted, and it is sometimes worthwhile to waste pea gravel undersize. Where the batching demand for various sizes of coarse aggregate is likely to be highly variable, as in many commercial plants, it will usually be necessary to finish screen each aggregate size individually to maintain balanced levels in the bins. With this type of individual finish screening, all undersize should be wasted. Coarse aggregate can be rewashed, if necessary, during finish screening. The water remaining on the material will not affect uniformity appreciably, unless batching is quite intermittent. In this case, the lesser of the two evils must be selected.

4. Without finish screening, stockpiling of coarse aggregate should be kept to a minimum. When stockpiles are necessary without finish screening, they should be built up in horizontal or gently sloping layers, not by end dumping (Fig. 1). Trucks and bulldozers should be kept off the piles; aside from breakage, such treatment may impair the cleanness of aggregate. A hard base should be provided to prevent contamination from underlying material in storage areas in continual use. Overlap of different material should be prevented with suitable walls or an ample distance between the storage piles. Cranes should be operated so as to avoid swinging buckets of one aggregate over another. Stockpiles of coarse aggregate, even under ideal conditions, tend to accumulate excessive fines which can be removed only by thorough rinsing and finish screening at the batch plant.

5. Storage bins should have the smallest practicable equal horizontal dimensions; round bins are preferable. To avoid accumulation of fines in dead storage areas, bottoms of bins should slope at an angle not less than 50 deg from the horizontal toward a center outlet. Bins should be filled by material falling vertically, directly over the outlet (Fig. 1). They should be kept as full as practicable at all times to avoid breakage and to minimize changes in grading as the material is drawn down.

6. Production or purchase of sand should be controlled to minimize variations in grading. Variation in fineness modulus of well graded sand should

be kept within a range of 0.20. In particular, the percentages of the finer sizes should be kept uniform and care should be exercised to avoid removal of fines essential to good grading and workability during washing operations. Well graded sand to produce plastic, workable concrete should have 15 to 20 percent finer than the No. 50 screen, of which not more than one-third should pass the No. 100 screen. Lean mixes are benefited by larger amounts of these fine sizes, while rich mixes are improved with smaller amounts of them.

7. No attempt should be made to blend two sizes of sand, by placing alternate amounts in stockpiles, in cars, or in trucks as loaded, or by other equally crude methods. Satisfactory results are obtainable where the fine aggregate is processed by blending the different sizes of sand as they flow into a common stream from regulating gates or feeders. This method can, however, produce very irregular results if adequate equipment and constant supervision and inspection are not employed. Separate batching of fine and coarse sand is preferable to any other method of blending, since it can produce consistently uniform results under the wide range of common plant and job conditions.

8. Varying amounts of water in aggregates contribute heavily to lack of uniformity in concrete consistency. Effort must be made to insure a uniform and stable moisture content in aggregate *as batched*. For this reason, aggregate with low absorption, from dry deposits, may preferably be prepared and handled dry when washing is unnecessary. Wind should not be permitted to separate dry sand (Fig. 1), and a $\frac{1}{8}$-in. maximum size may be required to prevent segregation of dry sand in piles and bins. Wet aggregate should be drained or mechanically filtered so that, as batched, it will have a moisture content that is reasonably uniform, and will be unaffected by interruptions requiring it to stand for several hours in the batch plant bin. This is particularly necessary for sand and small sized coarse aggregate. Stockpile drainage may take 48 hr or more to reduce moisture in sand to a uniform and stable amount. The time required depends on the grading and particle shape of the sand. Mechanical equipment for filtering unstable moisture from sand apparently is becoming available, and its use should be investigated and encouraged. Tests indicate that moisture up to at least 6 percent and possibly 7 or 8 percent will be adequately stable in concrete sand, depending on batching schedules and sand grading.

9. Much of the variability in promptly delivered ready-mixed concrete is the direct result of many unpredictable changes in source, character, grading, and moisture content of the aggregate supply. Precautions recommended in these paragraphs concerning aggregates apply to all concrete production, but are particularly applicable to ready-mixed concrete operations. Finish screening at the batching plant will eliminate in one operation the principal difficulties with coarse aggregate: stockpile segregation and contamination, careless loading to and from stockpiles, and variable overlap of aggregate sizes. Insistence on moisture in sand being less than a proved practical limit of stability, and use of moisture meters to indicate variations of moisture in sand as batched, will remove most of the remaining sources of

variation. The use of aggregates from sources of proved inherent quality, together with these provisions for assuring uniformity as batched, will result in a rapidly diminishing demand for costly isolation and separate use of pretested materials.

10. Samples representing the various sizes of aggregates *as batched* should be secured through facilities provided for that purpose at the batching plant, and sampling and testing should be scheduled as necessary to assure and confirm uniformity.

III. MEASUREMENT

General requirement

1. For reasons mentioned in Section II(1), materials should be handled, and measuring operations should be performed, in such a manner that satisfactory reproducibility of the selected batch assembly and aggregate grading is obtained in each batch, batch after batch. Batching equipment should be capable of performing accurate measurement. *The objective is uniformity and homogeneity of the concrete produced.* When aggregate is uniform, uniformity of batching is indicated by uniformity of unit weight, and strength tests from different batches supposedly the same. Moreover, for most uniform results, the mix should be proportioned from workable average gradings of aggregates in use, not from a single test.

2. Modern specifications issued by state and federal agencies contain detailed requirements for manual and automatic weigh batching equipment for cement and aggregates. The present equipment on the market, when in good mechanical condition, will ordinarily operate within the approved tolerance in weighing ability. Most recommended tolerances will not exceed 1 percent for cement and water, and 2 percent for each aggregate, or 1 percent for aggregates weighed cumulatively. Operation should be required within 1 percent accuracy for cement and 2 percent accuracy for aggregate, especially with automatic equipment. Cement consumption through automatic equipment should not vary more than ½ percent from the amount intended. Batching equipment should be insulated from plant vibration. If vibration from finish screening or mixing proves objectionable, weighing equipment should be supported independently. With good inspection and capable plant operators on the job, new equipment need not necessarily be required.

Batching aggregate

3. Factors affecting choice of proper weigh batching equipment are: (1) size of job, (2) rate of production, and (3) standards of batching performance. Available equipment falls into three general categories: manual, cumulative (semi-) automatic, and individual (full) automatic. Bins for aggregate in all types of batch plants should be sufficiently large, and so shaped, that different materials will remain apart and be handled in accordance with Fig. 2. Precautions previously recommended in Section II for keeping aggregate uniform should be followed prior to batching, in all types of batch plants. All weigh

batchers should be arranged to permit ready access for obtaining good representative samples of the aggregate as batched. Short conveyor belts, sometimes used to carry aggregate to and from individual batchers, provide an excellent facility from which to obtain representative samples of the aggregate as batched.

4. *Manual batching*—Manual batching includes wheelbarrow and platform scale setups which may be satisfactory and economical for jobs using less than about 1000 cu yd of concrete. As the size of the job increases, the cost of such hand batching soon justifies a manual weigh-batching installation. With this equipment, aggregate sizes are usually weighed cumulatively, and all operations are manual, including opening and closing of all bin and hopper gates. Manual plants may be acceptable for jobs up to about 5000 cu yd depending on job requirements. For jobs up to this size, coarse aggregate from stockpiles should be finish screened at least on sloping stationary screens to remove excess dirt and small undersize, limiting significant undersize to not more than 3 percent. If the material is damp, slotted openings in such screens are more effective.

5. *Cumulative automatic batching*—Cumulative automatic aggregate batching equipment automatically closes power-operated gates supplying a cumulative weigh batcher. Setting of batch weights, release of the weighed batch to the mixer, and push button starting of each automatic weighing cycle are manual. This plant provides greater accuracy and greater speed for a higher production schedule than manual batching. Graphic recorders of batching operations can be used with this equipment and should be required when good plant records are desired. The cumulative automatic plant should be required on jobs ranging in size from 5000 up to 30,000 or even 100,000 cu yd, depending on the rate and period of production, and, with recorders, in ready-mixed concrete plants producing 30,000 cu yd or more per year. For jobs of this size, including ready-mixed concrete plants, coarse aggregate should be finish screened over the batching plant on horizontally operating vibrating screens and should pass directly to the batch plant bins. For exceptions and discussion see Section II(3).

6. *Individual automatic batching*—Individual automatic aggregate batching provides a separate scale and batcher for each size of aggregate and cement; push button selection of the mix desired from a number of preset proportions; automatic batching that can be started automatically or manually as desired as soon as the individual batchers are discharged, and not before; simultaneous batching of all ingredients which saves time; batcher discharge gates that close as soon as batchers are empty, and not before; and automatic recording of all aspects of the batching operation including quantities of aggregates, cement, water, and admixtures, and a continuous record of batch-to-batch consistency.[4] Such plants provide the maximum in speed with best available uniformity. They are usually used to batch mass concrete for large dams and other heavy construction and should be used for other jobs requiring more than 100,000 cu yd, and for ready-mixed concrete plants producing more

than 100,000 cu yd of concrete per year. For jobs of this size, including the ready-mixed concrete plants, coarse aggregate should be finish screened over the batching plant on horizontally operating vibrating screens and pass directly to the batch plant bins. For exceptions and discussion see Section II(3).

7. Weigh batchers should be charged from easy operating clamshell or undercut radial type bin gates. Power operated bin gates are used with automatic batchers and may be used with manually operated batchers. Gates charging automatic batchers should be arranged to operate with a suitable "dribble" to add small increments after closely approaching the quantity desired in the batcher, unless other devices of proved ability to meet requirements for accuracy are provided.

Batching cement

8. Sacked cement for jobs batched manually should be measured in units of not less than 1 sack, unless the fractional bags are weighed. Cement for larger jobs should be handled in bulk and weighed for each batch with automatic equipment. Bulk cement should not be weighed manually. Cement should not be weighed with the aggregate; separate automatic weigh batching should be provided. All batchers should provide free access for testing and inspection to see that they discharge quickly and completely, and be equipped to permit removal of a sample at any time. Circular bins and batchers are preferable; at least they should have smooth metal sides with rounded corners. Cement batchers should be equipped with a vibrator to aid in securing complete discharge of the batch, and should be arranged so that the loading cycle cannot start again as long as cement remains in the batcher. Dry, low pressure (3-5 psi) air should be introduced in large volume through air pads to loosen cement which has settled tightly in storage bins. Controlled screw conveyors, rotary vane feeder units, or other effective devices that will permit a precise cutoff by the automatic control, should be used to charge cement weighing equipment.

9. Bulk cement storage should include a receiving bin for each type of cement in use on the job or in demand from a ready-mixed concrete plant. Each should have a separate conveyance to the cement weighing equipment or to a small supply bin for that equipment. Considerable care must be exercised to avoid putting cement into bins other than the one assigned to that brand or type.

10. If a pozzolanic or a secondary cementitious material is used, the foregoing recommendations for batching cement should be followed for handling, storing, and batching the additional material. Such material should be batched separately if not used from sacks, and separate screws and elevators for handling it should be used to insure that it is put only into the bin assigned to it. If either cement or added material creates a dust nuisance in the plant, effective means are available and should be employed to correct it.

11. Loss should be prevented whenever bulk cement is transferred. Free fall of cement from batchers should never be permitted. When loading the

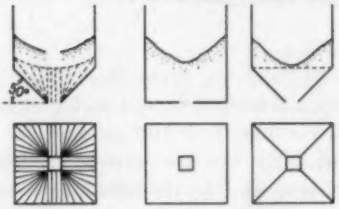
separate container for cement which should be provided in portable batch compartments, loss may be minimized by enclosing the discharging cement in a narrow canvas boot of such length that its end may be quickly buried in the discharged cement (Fig. 2). The boot may thus serve as a tremie, or to control the flow by kinking or doubling back the boot at the lower end. The boot may then be lifted, emptied, and transferred to the next compartment. A telescopic rubber hose drop-chute from the cement batcher is also available for this purpose. If a separate container is not provided for the cement in each batch compartment, or the cement is not enclosed in the aggregate, tarapulin covers should be required during transportation. For placing bulk cement into a batch hopper for a stationary mixer, a pipe of suitable size to hold the cement should extend from the batcher discharge to a level near the bottom of the hopper. After covering the end of this with aggregate, the cement may be discharged. It will then enter the mixer without loss of dust and will be well distributed through the entering aggregate.

Water measurement

12. Larger jobs justify the recommendation that water measurement for concrete mixed in pavers and other portable mixers be made with automatic meters or automatic vertical cylinder center-siphon discharge type measuring tanks. Either of these devices or automatic weigh batchers should be used in central mixing or batching plants. Any method used should be capable of routine measurement within an accuracy of 1 percent under all operating conditions. Tanks, except in the form of a vertical cylinder with center-siphon discharge, should be permitted only as an auxiliary part of the automatic meter or weigh batcher and not as a means of measurement. Truck mixers should be equipped with water measuring equipment meeting these requirements and capable of injecting the water deeply into the drum where it will most promptly become well distributed in the batch. Equipment for water measurement should be fitted with such valves and connections as are necessary to conveniently divert the water measured for a batch, so that accuracy of measurement or adjustments can be quickly verified.

13. Measurement of the correct amount of water for concrete is so intimately related to the amount of water in the sand, and particularly to variations in sand moisture, that these recommendations would be incomplete without mention of equipment now available for indicating the amount and variation of moisture in sand as it is batched.[10] Although the accuracy of such equipment is less than that of laboratory methods, it is sufficient when properly operated to indicate accurately the general magnitude of increases or decreases in sand moisture from which appropriate adjustments can be made in the measured water in time to keep consistency reasonably constant. Where properly used, this equipment has proved to be a large factor in improving the uniformity of concrete, and it should be used on jobs and in ready-mixed concrete plants large enough to warrant automatic equipment. Means should be provided for quickly adjusting water and sand weights to

UNIFORMITY OF CONCRETE IS AFFECTED BY THE ARRANGEMENT OF BATCHER-SUPPLY BINS AND WEIGH BATCHERS



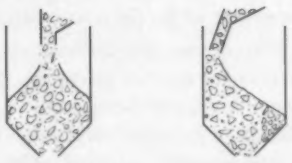
CORRECT

Full bottom sloping 50° from horizontal in all directions to outlet with corners of bin properly rounded so that all material moves toward the outlet.

INCORRECT

Flat bottom bins or those with any arrangement of slopes having corners or areas such that all material in bins will not flow readily through outlet without shoveling.

SLOPE OF AGGREGATE BIN BOTTOMS



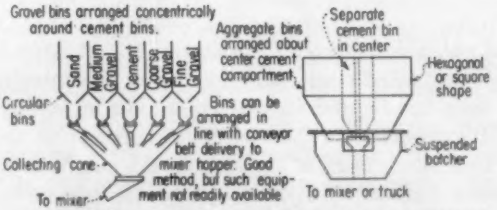
CORRECT

Material drops vertically into bin directly over the discharge opening permitting discharge of more generally uniform material.

INCORRECT

Chuting material into bin on an angle. Material falling other than directly over opening not always uniform as discharged.

AGGREGATE BIN FILLING

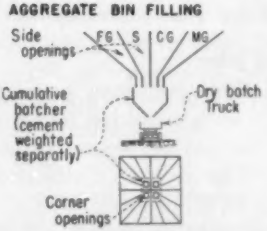

Gravel bins arranged concentrically around cement bins.
Sand
Medium Gravel
Cement
Coarse Gravel
Fine Gravel
Circular bins
Collecting cone
To mixer
Aggregate bins arranged about center cement compartment
Bins can be arranged in line with conveyor belt delivery to mixer hopper. Good method, but such equipment not readily available
Separate cement bin in center
Hexagonal or square shape
Suspended batcher
To mixer or truck

GOOD ARRANGEMENT

Automatic weighing of each ingredient in individual weigh batchers, discharging through collecting cone directly into mixer. Discharge of cement batcher controlled so that cement is flowing while aggregate is being delivered. Batchers insulated from plant vibration. Will permit overload correction.

ACCEPTABLE ARRANGEMENT

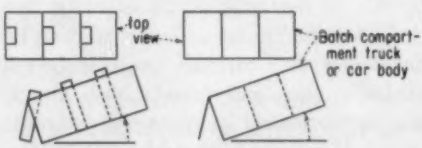
Aggregate automatically weighed separately or cumulatively. Cement weighed separately. Batchers insulated from plant vibration. Weight recording equipment plainly visible to operator. Proper sequence of dumping materials necessary. Avoid aggregate constantly flowing over top of material in bins. Will not permit correcting overloads.



THESE ARRANGEMENTS LIMIT UNIFORMITY

Either of above close groupings of bin discharges which cause long slopes of material in bins result in separation and impaired uniformity

PROVISION FOR CEMENT IN DRY BATCH COMPARTMENTS



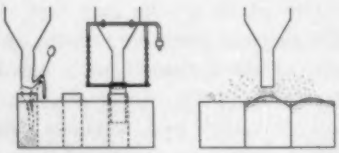
CORRECT

Provides separate compartments of suitable size and depth attached to and operating with each batch release gate.

INCORRECT

Cement dumped on or within aggregate may be blown away, partially prehydrated, or may slide into another batch in dumping.

LOADING BATCHED CEMENT



CORRECT

Fall of cement controlled by enclosing in kinked canvas drop chute or telescopic flexible hose tremie.

INCORRECT

Free fall of cement into batch car or truck causes waste, and overlap of batches is common.

BATCHING EQUIPMENT

Fig. 2—Correct and incorrect batching procedures

compensate for moisture variation. Uniformity in measurement of total mixing water is indicated by the uniformity of consistency of the concrete.

Measurement of liquid admixtures

14. Use of an air-entraining admixture, and on occasion certain other admixtures for special purposes, has become a worthwhile and widely accepted practice. Dosages being relatively small compared to the amount of other ingredients, comparatively delicate dispensing devices are necessary. For air-entraining agents, calcium chloride solutions, and water-reducing retarders accuracy of measurement within 5 percent of the prescribed dosage is satisfactory. Usually adequate equipment is available from vendors of admixtures, but it is essential that it be maintained in a clean and accurate working condition. Moreover, provision should be required on each dispensing device for routine diversion of a measured dosage into a small container or graduate for daily verification of the batch quantity.

Small jobs

15. For small jobs, placing of concrete is likely to be irregular. It is preferable for a small job to use well controlled ready-mixed concrete or centrally dry-batched aggregates rather than handle and measure materials at the job site. The dry batch size should correspond to the maximum capacity of the mixer to be used at the site. Fractional bags of cement should not be used unless they are weighed for each batch. Use of simple water measuring equipment that is accurate, positive, and dependable should be required.

General

16. Emphasis should be placed on correct procedure in operations other than batch weighing if readily obtainable standards of uniformity are to be maintained. Care should be exercised to insure that accurately assembled batches arrive as uniform batches in the mixer. The following sources of error should be avoided: (1) overlap of batches in loading and discharging multiple batch trucks and cars, (2) loss of material in transferring batches to the skips of portable mixers, and (3) loss or "hang-up" of a portion of one batch, or its inclusion with another when dry batches are transferred by belts and hoppers. Incorrect quantities in each assembled batch should be strictly avoided, and methods should be approved or discarded accordingly. Batches should be protected from rain. Truck beds should be clean for this use.

IV. MIXING

Essential requirements

1. Thorough mixing is essential for uniformity of workability, strength, and other concrete properties. Equipment and methods for mixing concrete should be those which will produce uniformity of consistency, cement and water content, and aggregate grading from beginning to end of each batch *as discharged.* For concrete of the highest quality, mixing equipment must be capable of handling concrete containing the largest aggregate and the lowest

slump which is placeable in the particular work and which can be readily consolidated by vibration. Mixing equipment that requires, for efficient operation, mixes containing smaller aggregate and larger proportions of sand, cement, and water than are needed for best results in the forms should not be used. Sufficient mixing and placing equipment should be provided so that the work may be kept alive and free from cold joints.

The mixer

2. Drum mixers should have a combination of blade arrangement and drum shape such as to insure an end-to-end exchange of the materials parallel to the axis of rotation, as well as a rolling, folding, or spreading movement of the mix over on itself as the batch is mixed. A counter current type of mixer may be used, in which blades revolving on vertical shafts operate counter clockwise in a large pan revolving clockwise and in which the batch can be easily observed and quickly adjusted for correct consistency. Whatever the mixer type, it should exhibit acceptable mixing performance [Section IV(10)] within a reasonable mixing period. Each mixer should be operated at the speed which gives the required mixing performance in the minimum time regardless of the speed designated by the manufacturer.

3. Any mixer leaking mortar, or causing waste of materials through faulty charging, should be taken out of service and kept out until satisfactory repairs and improvements have been made. Inner surfaces of mixing equipment should be kept free of hardened concrete and mortar. Worn blades which materially decrease mixing efficiency should be replaced.

Batch size

4. Batch size should not exceed the manufacturer's guaranteed capacity or 10 percent more than the rated capacity. When, to shorten time of discharge or to improve mixing performance, it is desired to retain a portion of the batch in excess of normal "hold-back," the amount of material held back should be considered in fixing the size of the batch in relation to the capacity of the mixer.

Charging operation

5. It is particularly important in charging stationary and truck mixers, that the solid materials be arranged in the charging hopper in such a manner that proportional amounts of each will be in all parts of the stream as it flows into the mixer. It is preferable, to avoid gumming, that cement not be charged separately; for good uniformity in truck mixing when this is necessary considerable additional mixing is required. Also considerable cement may be lost if it is not enclosed within the other materials or properly shrouded during charging. Care should be taken to see that the water is released first and continues to flow while the solids are entering the mixer. Discharge pipes of all water batchers must be of such a size and so arranged that the flow into the mixer will be completed within the first 25 percent of the mixing time and will be delivered well inside the mixer where it will be mixed quickly

with the entire batch. Where meters are used, auxiliary tanks, into which the batch quantity of water can be measured in advance, may be necessary.

Mixing time

6. Mixing time required depends to a large extent on the efficiency of the mixer used. Experience has shown that mixing time for a 1 cu yd or smaller mixer, when mixing gravel concrete of medium consistency containing more than average cement content, should not be less than 1 min under the best conditions. For concretes of less than average cement content, mixed to drier consistencies, or containing an unusually harsh-mixing aggregate, the mixing time should be increased, sometimes as much as 50 percent.

7. For job mixers larger than 1 cu yd capacity, particularly when mixing large maximum size aggregate concrete low in cement content and low in slump, a minimum net mixing time of 1 min plus ¼ min for each additional cubic yard of capacity is often required. Regardless of the manner of charging, the total net mixing time for any mixer should be the time required to bring the batch within the requirements of the mixer performance test [(Section IV(10)] when the mixer is charged and operated in accordance with practice established as routine for the job.

8. The mixing period should be measured from the time when all the solid materials are in the mixer. No portion of the time required for discharging should be considered a part of net mixing time. Slight overmixing is not objectionable unless it results in loss of entrained air or in requirement for additional water; however, mixing equipment should be so arranged that it can be stopped and started under full load.

9. For maintenance of proper and uniform mixing time, a batch timer, with an audible indicator and an automatic lock which will not release the discharge mechanism until the completion of a preset mixing time, is an actual necessity on large mixers and often a useful device on ½ cu yd or smaller mixers. For use on all truck mixers reliable "reset counters" are recommended.

Mixing performance

10. Mixer performance in intermingling constituents of concrete in all types of mixes should be such that prior to discharge at the end of the prescribed mixing period, two samples, one taken at the front, and one taken at the rear, will not exceed the following limits of uniformity (Reference 4, p. 447):

(*a*) Unit weights of air-free mortar from the two samples will not vary more than 0.8 percent from the average of the two mortar weights.

(*b*) Weights of coarse aggregate retained on a No. 4 screen from the two samples will not vary more than 5 percent from the average of the two weights of coarse aggregate.

The ratio of coarse aggregate to mortar should appear to be uniform in all parts of the mixer on visual inspection and should be borne out by subsequent observation of the concrete after discharge.

11. Another important aspect of mixing performance is batch-to-batch uniformity.

(*a*) Although this depends largely on the uniformity of materials and their measurement for each batch, uniformity of consistency depends largely on the skill and continued attention of the mixer operator. If precautions here recommended are taken, it should be practical to require that batch-to-batch variation in slump should be maintained within a range equal to one-half the average slump.

(*b*) Uniformity of air content is an aspect of mixer performance in that it depends in part on uniformity of consistency. Air content should not vary more than 1 percent above or below the average percentage of air selected or specified.

(*c*) Uniformity of mixer performance also depends on uniformity of temperature, inasmuch as temperature variations cause variations in slump and in air content. When certain temperatures are desired, as in specifications for cold or hot weather concrete, the temperature of concrete as mixed should not vary much more than 5 F above or below the average temperature selected or specified.

(*d*) A recording consistency meter not of the torque type has definite advantages on 2 cu yd or larger stationary mixers, not only in the maintenance of uniform consistency, but also because it provides a permanent record of batch count, type of batch, mixing time, and retempering. Another important aid to maintaining uniform consistency is visibility of the concrete during mixing; provision should be made for conveniently observing the mixing action in mixing equipment.

Retempering

12. Under careful supervision and followed by additional mixing equal to half of minimum required mixing time, a small increment of water may sometimes be added to improve workability of delayed batches in plant or job mixers, provided the maximum allowable water-cement ratio is not exceeded.[7] Indiscriminate retempering or addition of water as desired to increase slump after concrete is first mixed should be prohibited. If slump loss or delays in placing are sufficient to result in serious loss of workability, other appropriate action should be taken. These matters are covered for truck mixers in Sections IV(18) and IV(20). Batches considerably higher in slump than specified, or batches in which there has been an obvious error in measurement, should be wasted from any mixer, unless proper corrections can be made promptly and the corrected batch thoroughly mixed.

Discharge operation

13. All types of mixers should be capable of ready discharge of concrete of the lowest slump which can be consolidated by vibration. In many massive or unformed placements, modern vibrators will readily consolidate concrete of a 1-in. slump.

14. Separation of coarse aggregate from the mortar, which commonly results on discharge from most plant and transit mixers, should be avoided by arrangement of the discharge as shown in Fig. 3, so that the concrete will fall vertically, not diagonally, into whatever container is to receive it.

15. Blade arrangement and discharge mechanism of all types of mixers, including agitating, shrink, and transit mixers, should be such that throughout the discharging operations the aggregate is well distributed from coarse

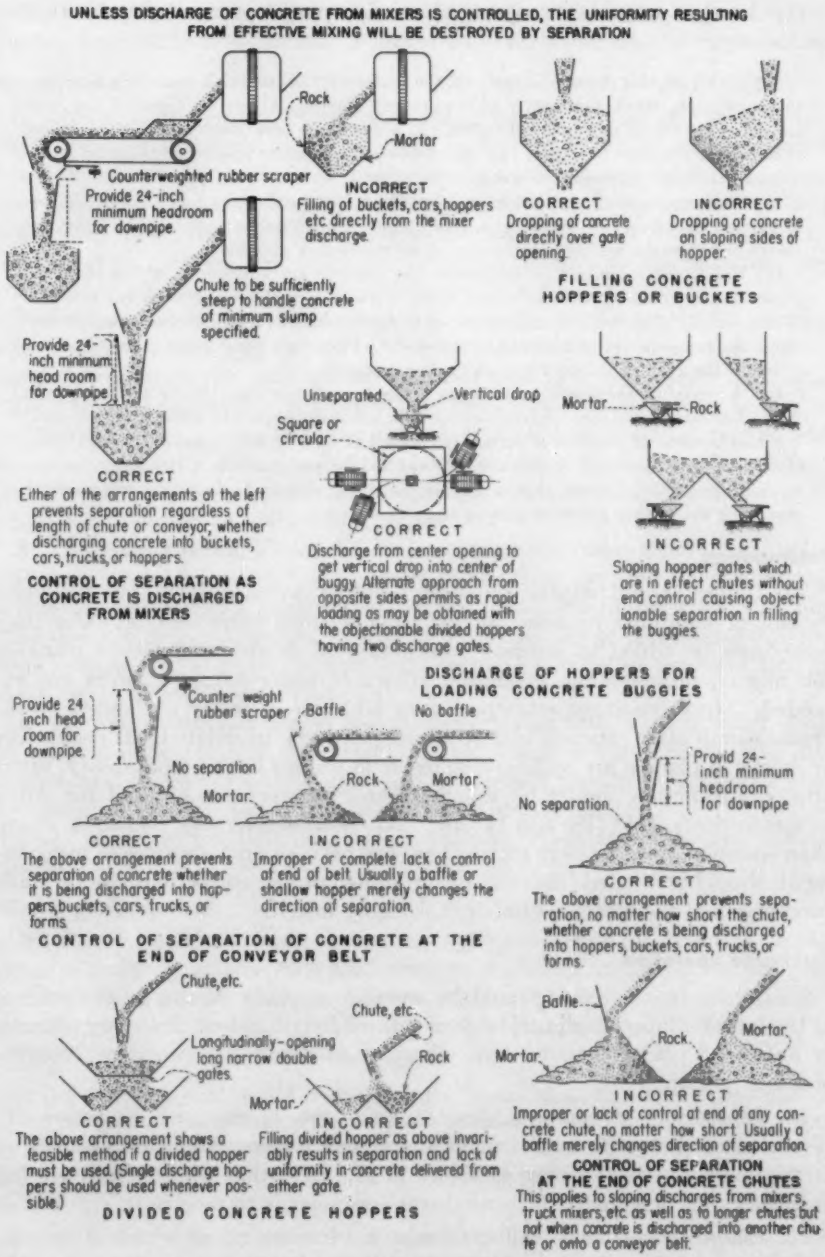

UNLESS DISCHARGE OF CONCRETE FROM MIXERS IS CONTROLLED, THE UNIFORMITY RESULTING FROM EFFECTIVE MIXING WILL BE DESTROYED BY SEPARATION
Rock
Mortar
Counterweighted rubber scraper
Provide 24-inch minimum headroom for downpipe.
INCORRECT
Filling of buckets, cars, hoppers etc. directly from the mixer discharge.
CORRECT
Dropping of concrete directly over gate opening.
INCORRECT
Dropping of concrete on sloping sides of hopper.
FILLING CONCRETE HOPPERS OR BUCKETS
Chute to be sufficiently steep to handle concrete of minimum slump specified.
Provide 24-inch minimum head room for downpipe
CORRECT
Either of the arrangements at the left prevents separation regardless of length of chute or conveyor, whether discharging concrete into buckets, cars, trucks, or hoppers.
CONTROL OF SEPARATION AS CONCRETE IS DISCHARGED FROM MIXERS
Unseparated
Vertical drop
Square or circular
CORRECT
Discharge from center opening to get vertical drop into center of buggy. Alternate approach from opposite sides permits as rapid loading as may be obtained with the objectionable divided hoppers having two discharge gates.
Mortar
Rock
INCORRECT
Sloping hopper gates which are in effect chutes without end control causing objectionable separation in filling the buggies.
DISCHARGE OF HOPPERS FOR LOADING CONCRETE BUGGIES
Provide 24 inch head room for downpipe
Counter weight rubber scraper
Baffle
No baffle
No separation
Rock
Mortar
CORRECT
The above arrangement prevents separation of concrete whether it is being discharged into hoppers, buckets, cars, trucks, or forms.
INCORRECT
Improper or complete lack of control at end of belt. Usually a baffle or shallow hopper merely changes the direction of separation.
Provid 24-inch minimum headroom for downpipe
No separation
CORRECT
The above arrangement prevents separation, no matter how short the chute, whether concrete is being discharged into hoppers, buckets, cars, trucks, or forms.
CONTROL OF SEPARATION OF CONCRETE AT THE END OF CONVEYOR BELT
Chute, etc.
Longitudinally-opening long narrow double gates.
CORRECT
The above arrangement shows a feasible method if a divided hopper must be used. (Single discharge hoppers should be used whenever possible.)
Chute, etc.
Rock
Mortar
INCORRECT
Filling divided hopper as above invariably results in separation and lack of uniformity in concrete delivered from either gate.
DIVIDED CONCRETE HOPPERS
Baffle
Rock
Mortar
Mortar
INCORRECT
Improper or lack of control at end of any concrete chute, no matter how short. Usually a baffle merely changes direction of separation.
CONTROL OF SEPARATION AT THE END OF CONCRETE CHUTES
This applies to sloping discharges from mixers, truck mixers, etc. as well as to longer chutes but not when concrete is discharged into another chute or onto a conveyor belt.

Fig. 3—Correct and incorrect methods of handling mixed concrete

to fine. Should the last fraction of the batch contain an excessive amount of coarse aggregate, this portion should be retained and mixed with the succeeding batch. Batch size in this case should be reduced by an amount corresponding to the quantity withheld. In some truck mixers this type of separation may be alleviated somewhat by reversing the direction of rotation for 10 or 12 revolutions prior to discharge. Coarse aggregate is in objectionable excess when that larger than ¾-in. exceeds by 15 percent the amount of that size batched.

Ready-mixed concrete

16. Ready-mixed concrete may be mixed in a central plant and transported to the job in agitating or nonagitating truck bodies, mixed in a truck mixer in transit to or after the job location has been reached, or partially centrally mixed and mixing completed in truck mixers en route to the job ("shrink-mixing"). Adequately equipped and well supervised ready-mixed concrete operations afford excellent opportunities for control of concrete quality.

17. Control of quality of ready-mixed concrete presents some problems not common to other concrete mixing operations. Certain conditions peculiar to ready-mixed concrete may require special attention. Recommendations in the preceding text relative to selection and proper handling of materials, accuracy of measurement, and adequate mixing apply whether the concrete is job mixed or ready mixed. Problems in ready-mixed concrete which may exist to a more troublesome degree generally originate in the indiscriminate addition of mixing water and breakdown in control of consistency. Concrete is generally regarded as having the correct consistency when it is adequately plastic and workable, readily placed, and not subject to objectionable segregation due to transportation and handling. Responsibility for the in-place quality of ready-mixed concrete is divided between the producer and the purchaser. Unless the consistency and addition of mixing water are properly controlled, serious impairment of quality can result from indiscriminate addition of water regardless of who ordered it. The job provided with inspectors who have the ability, the will, and the authority to exercise proper control, can be serviced satisfactorily by any dependable ready-mixed concrete operation. When such inspection is not available (the case on the average job serviced with ready-mixed concrete), the producer should take the initiative to encourage proper job control and procedures and should be prepared to furnish assistance and advice to achieve this end. Breakdown of job control may often result in undeserved discredit to the individual concrete producer and to the ready-mixed concrete industry as a whole. Such difficulties as poor preparation and lack of organization to receive ready-mixed concrete when it is delivered, unrestricted use of retempering water, common placement abuses, and inadequate curing which concrete subsequently receives, may all be encountered to some degree when meaningful inspection is lacking. None of these abuses experienced with, and often attributed to, ready-mixed concrete could exist if such performance was not accepted de

facto by a large segment of the construction industry, including owners, architects, and engineers.

18. Because concrete stiffens, water required for proper consistency is affected by length of haul, amount and rate of mixing, climatic conditions, characteristics of ingredients, time required for unloading, and other less prevalent considerations. In cool weather, with short hauls, and when delivery is prompt, particularly on large, well inspected, continuous placements, negligible difficulty arises from loss or variation in slump, higher mixing water requirements, and discharge, handling and placing problems. Conversely, when the rate of delivery required is irregular, haul distances are long, and placements are small, slow, and scattered, or weather is warm, the problems encountered in striving for uniformity and quality can be serious. Considerable loss of slump in such circumstances is too often compensated for by generous additions of water at the start, so that slump will be ample when the last of the load is discharged. Such additions of water should be regarded with disfavor beyond compensation for 1 in. of slump loss. Under the difficult conditions mentioned earlier in this paragraph, precautions should be taken to minimize loss of slump by expediting delivery and placement on arrival, eliminating delays and, in warm weather, by using a retarder. The concrete should be kept as nearly in the 70 to 80 F temperature range as practicable by using cold mixing water or ice, avoiding the use of hot cement, shading and sprinkling aggregate, and by painting the equipment white. In such situations when loss of slump cannot be offset by these measures, the difficulties can be minimized by adding all the water and doing all mixing at the job site, or by supplying a suitable mixer at the forms with centrally dry-batched materials.

19. A major factor in transit-mixed concrete is that the control is less centralized. It, therefore, requires rigid control of all factors governing slump because of the difficulty of judging consistency until concrete is discharged at the point of delivery. Variations in consistency may be minimized by:

(*a*) Handling aggregates in such a manner as to reduce variations in grading and moisture content to a practical minimum.

(*b*) Verifying that mixers do not contain variable amounts of water before batching and absolute control of the added water to prevent indiscriminate and unauthorized additions.

(*c*) Regulation of the number of revolutions at mixing and at agitating speed to reasonable uniformity from batch to batch.

(*d*) Coordination between batching plant and point of placing to assure a minimum quantity of concrete arriving before a change order goes into effect, or before concrete on hand can be placed. Two-way radio between trucks and batch plant is used to great advantage in this connection by many operators of ready-mixed concrete plants and is a recommended solution to this important problem of coordination.

(*e*) Responsible technical supervision of the entire operation.

20. It is desirable, but frequently not practicable, to completely control mixing water at the batching plant. It may be measured there successfully when the amount of moisture present in the aggregate is accurately known

and taken into acount. Under these conditions the correct amount of added mixing water can be batched to the mixing water tank or the mixer drum. In cases where sand moisture is variable and cannot be rapidly and accurately determined, and in situations causing loss of slump greater than 1 in., some or all of the added mixing water should be withheld until the mixer arrives at the forms. With the required water added, the concrete requires an additional 20 to 30 revolutions of the drum at mixing speed to thoroughly incorporate added water into the mass. While every effort should be made to have concrete batches arrive at the job site at a suitable consistency, it is possible to make adjustments after arrival if proper provisions are made in specifications and on the job. Adjustments after the concrete reaches the forms, however, delays the work and interferes with orderly routine. Moreover, too often, it is in "adjustments" that control is lost.

21. Transit mixers should be operated at the mixing speed and number of revolutions found by performance tests [Section IV(10)] to accomplish thorough mixing and be capable of operating at a suitable lower speed for agitation. In transit mixing and shrink mixing, no more revolutions of the drum should be made at mixing speed than are necessary for concrete to meet the mixer performance requirements given in Section IV(10). Mixing for agitation only should be at agitator speed. The batch volume should not exceed 63.25 percent of the gross volume of the drum. Agitator trucks should not be loaded in excess of the quantity recommended by the manufacturer, as this may prevent proper agitation. Moreover, agitation of mixed concrete should be kept to the minimum required for homogeneity at delivery; usually 10 revolutions at mixing speed, or agitation during the last ½ mile before delivery, is sufficient.

22. Bodies of nonagitating delivery trucks should be smooth, preferably streamlined, water-tight metal containers equipped with gates that will permit control and ready discharge of the concrete. Covers should be provided against the weather when required. Slump tests of individual samples, taken at approximately the one-fourth and three-fourths points of the load as discharged, should not differ by more than 1 in. If the foregoing requirement is not met, lower slump, air entrainment, lengthening of mixing time, or smaller loads may correct the difficulty.

23. In devoting a separate section to ready-mixed concrete, it is not the intention to imply that other aspects of these recommendations are not applicable to this operation. In many cases they are applicable and accordingly are recommended as well for ready-mixed concrete operations. Specifications for concrete work should include requirements herein recommended for ready-mixed concrete, and inspection should be established which is authorized to make sure that they are followed as to aggregate preparation, batching plant, and performance. Many operators of ready-mixed concrete plants are equipped and willing to provide such service and to accept responsibility for furnishing concrete as specified. It should be clearly recognized that suppliers of ready-mixed concrete can be held responsible only for those ma-

terials and operations which they control. Ready-mixed concrete is dependable when it is dependably handled.

V. PLACING

Basic requirements

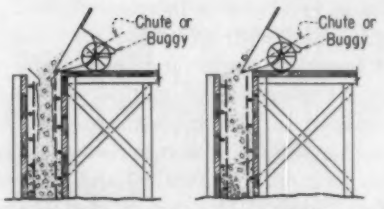
1. Foremost among basic requirements is that at all points from mixer to concrete in place, only those methods and arrangements of equipment should be used which will reduce to a minimum any separation of coarse aggregate from the concrete. For highest quality of concrete *in place*, equipment should be selected for its ability to handle concrete of the most advantageous proportions that can be readily consolidated by means of vibration after placement. Equipment should be capable of expeditiously handling and placing concrete of such a proper consistency, grading, and maximum size of aggregate, at the rate most favorable to good quality and workmanship. No part of any concrete handling and placing equipment should be accepted if it requires, for efficient operation, mixes containing larger proportions of sand, cement, or water, or smaller coarse aggregate than are practicable in mixes readily consolidated by vibration.

2. Sufficient mixing and placing capacity should be provided so that the work may be kept alive and free from cold joints. Formed concrete should be placed in horizontal layers not deeper than 2 ft, avoiding inclined layers and inclined construction joints. To get a monolithic and sightly placement, it is important that each layer be shallow enough so as to be placed while the previous layer is still soft and that the two layers be vibrated together. Concrete should not be allowed or caused to flow horizontally or on slopes in the forms. Concrete placing on a slope should begin at the lower end of the slope and progress upward thereby increasing compaction of the concrete. Pneumatic placement of concrete should be avoided unless usual high velocity discharge is reduced to a point where no separation and scattering of the concrete occurs.

Aggregate separation

3. The most important consideration in handling and placing concrete is that of avoiding separation of coarse aggregate from the concrete. Scattered individual pieces of separated coarse aggregate are not objectionable because they are readily enclosed and consolidated into the concrete as it is placed and vibrated around them. Obvious groups and clusters of separated coarse aggregate are objectionable; they should be scattered before concrete is placed over them otherwise they may cause serious imperfections in the finished work. Particular attention should therefore be paid to the tendency for objectionable separation to occur at the ends of chutes and conveyor belts, at hopper gates, and at all other points of discharge so that uniformity and homogeneity of concrete in place and good workmanship, will be assured (see Fig. 3). It is a common fallacy that separation occurring in handling will be eliminated in the course of other operations. Separation must be prevented—not corrected after its occurrence.

CONCRETE WILL SEPARATE SERIOUSLY UNLESS INTRODUCED INTO FORMS PROPERLY



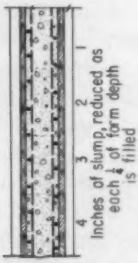
CORRECT

Discharge concrete into light hopper feeding into light flexible drop chute. Separation is avoided. Forms and steel are clean until concrete covers them.

INCORRECT

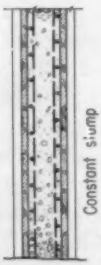
To permit concrete from chute or buggy to strike against form and ricochet on bars and form faces causing separation and honeycomb at the bottom.

PLACING CONCRETE IN TOP OF NARROW FORM



CORRECT

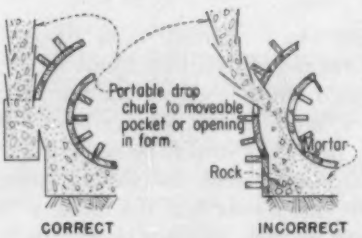
Necessarily wetter concrete at bottom of deep narrow form made drier as more accessible lifts near top are reached. Water gain tends to equalize quality of concrete. Settlement shrinkage is minimum.



INCORRECT

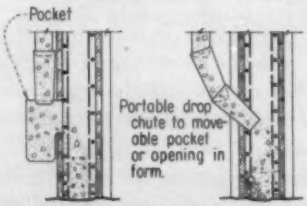
To use same slump at top as required at bottom of lift. High slump at top results in excessive water gain with resultant discoloration, loss of quality and durability in the upper layer.

CONSISTENCY OF CONCRETE IN DEEP NARROW FORMS



CORRECT

INCORRECT



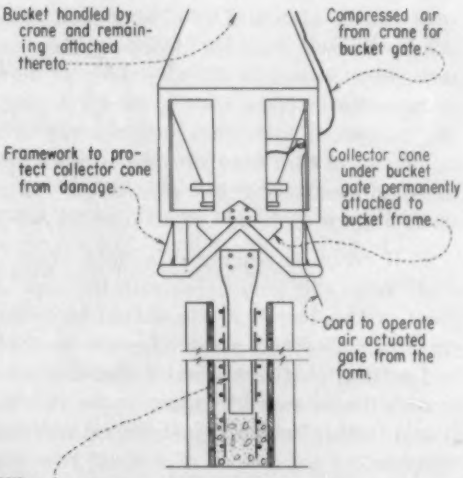
CORRECT

Drop concrete vertically into outside pocket under each form opening so as to let concrete stop and flow easily over into form without separation.

INCORRECT

To permit high velocity stream of concrete to enter forms on an angle from the vertical. This invariably results in separation.

PLACING IN DEEP OR CURVED WALL THRU PORT IN FORM


Bucket handled by crane and remaining attached thereto.
Compressed air from crane for bucket gate.
Framework to protect collector cone from damage.
Collector cone under bucket gate permanently attached to bucket frame.
Cord to operate air actuated gate from the form.
Flexible drop chute attached to collector cone. Chute collapses flat when no concrete is dropping, allowing it to be used for the smallest size aggregate as well as being large enough for the largest.

PLACING CONCRETE IN DEEP NARROW FORMS

Fig. 4—Correct and incorrect methods of placing concrete in forms

4. Important in arranging equipment to prevent separation is the provision that the concrete shall *drop vertically* into the center of whatever container receives it. The importance of this increases greatly with increases in slump, maximum size, amounts of coarse aggregate, and with reductions in cement content. To protect tie rods, spacers, embedded features and form surfaces and to prevent displacement of reinforcement, concrete falling in forms where it may damage these items should be confined in a suitable drop chute to within a few feet of concrete in place. Whether such a fall is into forms or into a hopper or bucket, the final portion of the drop must be vertical and without interference if separation is to be avoided as shown in the various sketches in Fig. 3 and 4. If forms are open sufficiently so that concrete can be dropped into them with little or no disturbance of rods, etc., it will usually be found that direct discharge into such forms is faster and saves the labor of placing, moving, and removing hoppers and drop chutes (Reference 4, p. 260). Moreover, where direct dumping is practicable otherwise, it results in little but unimportant scattered separation, whereas fixed drop chutes often cause considerable separation due either to flowing or stacking of the concrete. Mortar coating of rods during placement should be of no concern up to the level concrete will be placed; above that, mortar should be kept from or cleaned from the steel.

5. In difficult cases of placing in deep, narrow, reinforced walls, good results can be obtained by closing each third space between 6-in. studs on one side to form a duct for dropping the concrete which then enters the form through holes cut in the sheathing at vertical intervals not greater than 4 ft as concrete in place rises to each opening. A pocket should be provided at the bottom of each duct, successively upward below each opening as shown in Fig. 4, so that concrete will stop and flow easily over into the forms with minimum scattering and separation. A good internal vibrator should be in operation on each side of an opening while concrete is entering the forms.

6. If concrete placing in a deep beam, wall, or column is intended to be continuous and monolithic with the slab above, a delay to provide for settlement of the deep concrete should be scheduled before placing the upper concrete in soffits and slabs. Length of the delay will depend on temperature and setting characteristics of the concrete but should be as long as will still permit the vibrator to again make the deep concrete plastic as it revibrates it and further settles it just before and during vibration of the soffit and slab concrete.

7. Where applicable, bottom discharge buckets are a superior method of handling and placing concrete in massive and other structures, provided:

(*a*) Separation is avoided in filling the buckets as indicated in Fig. 3.

(*b*) Only one or more complete mixer batches are placed in the bucket unless the buckets are filled from a hopper in which there is no separated concrete.

(*c*) They are capable of discharging concrete of the lowest slump which can be consolidated by vibration as placed.

(*d*) They discharge concrete fast enough and from a point high enough so that the concrete will not stack and separate.

(*e*) They are of a size, and may be discharged in a manner and with such frequency, that concrete can be placed in approximately horizontal layers while the previous layer is still soft.

(*f*) Successive batches are so placed as to afford opportunity for thoroughly consolidating the concrete by means of internal vibrators.

Vibration

8. Vibration is the most effective known method of obtaining consolidation of newly placed concrete. Among methods of vibrating concrete, internal vibration is generally most effective. Vibration not only results in superior workmanship and appearance, but it also permits successful routine use of concrete that is less wet than is necessary for hand working and therefore more durable and of higher quality. Advantages and importance of vibration for consolidation of newly placed concrete are so well established that general use of vibration for this purpose is recommended.

9. For detailed recommendations concerning vibration of concrete, Reference 8 is suggested.* Briefly, equipment for vibration should have adequate power, and be of high frequency, rugged, and reliable. Operators should be experienced, competent, dependable, and energetic. Ample standby units and parts as well as systematic servicing should be provided. Vibrators should not be used to cause concrete to move more than a short distance laterally, otherwise fine wet material runs ahead and separates from the coarse aggregate. Vibrators should not be used in wet concrete because the rock sinks and the fines and water may come to the top; when this happens or is likely, when vibrators are used, the concrete should be mixed with appreciably less slump. Vibrators should be inserted and withdrawn at many points, from 18 to 30 in. apart, for short periods usually from 5 to 15 sec in preference to insertion at wider intervals for longer periods. Systematic spacing of these insertions should be established to insure that no concrete remains unvibrated (Fig. 5).

10. Inadvertent or intended revibration of concrete, or of steel embedded in it, is beneficial provided the concrete becomes momentarily plastic again during the revibration. As long as a running vibrator will sink into the concrete of its own weight, it is not too late for the concrete to benefit by revibration. Both compressive strength and bond with reinforcing are usually increased by late revibration, and separation, which might occur due to settlement shrinkage, is reconsolidated. Ordinarily there is little likelihood of damage to newly hardened lower lifts from revibration above, because the vibrators will not sink of their own weight into such concrete. Nor is vibration of steel which penetrates lower lifts likely to be detrimental, apparently because of the damping effect of deep embedment; at least no evidence of such damage is known.

*At this writing a revision of this recommendation is in preparation and should be used when available.

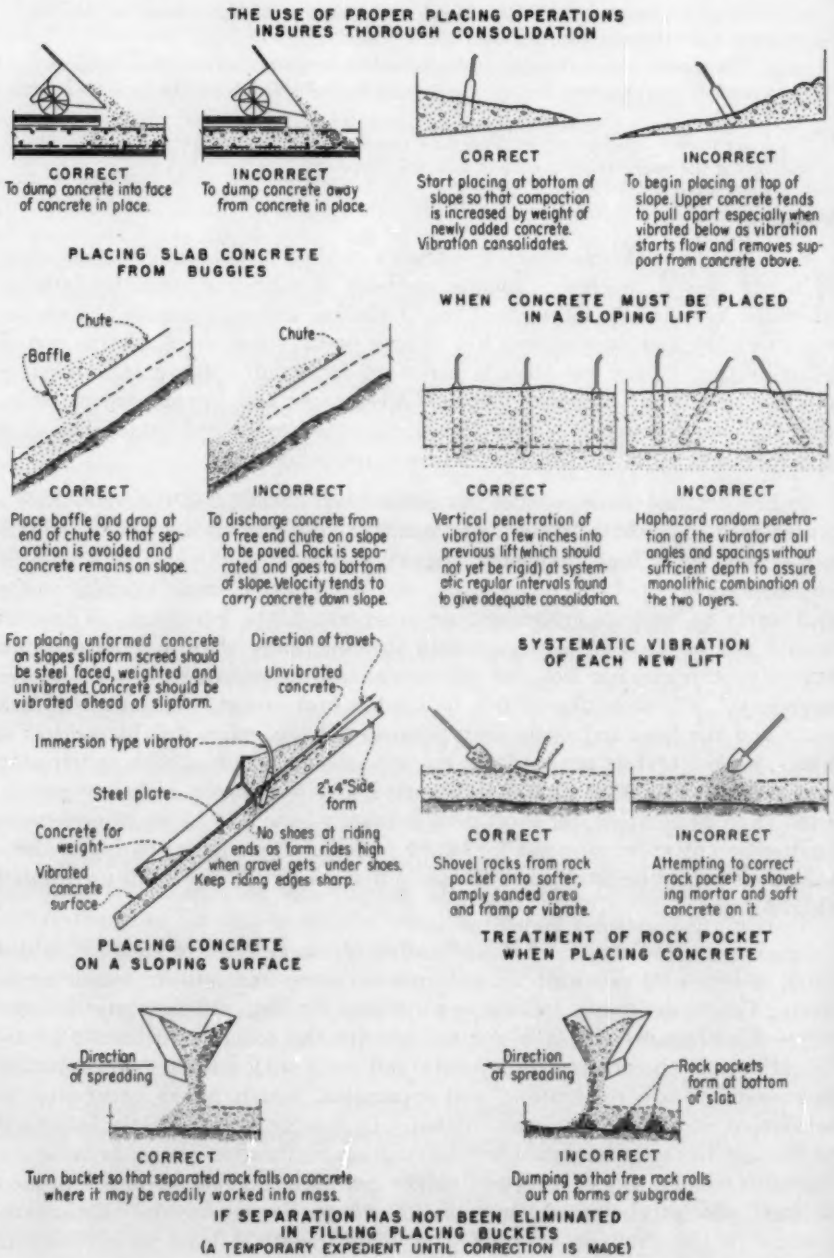

THE USE OF PROPER PLACING OPERATIONS INSURES THOROUGH CONSOLIDATION
CORRECT
To dump concrete into face of concrete in place.
INCORRECT
To dump concrete away from concrete in place.
CORRECT
Start placing at bottom of slope so that compaction is increased by weight of newly added concrete. Vibration consolidates.
INCORRECT
To begin placing at top of slope. Upper concrete tends to pull apart especially when vibrated below as vibration starts flow and removes support from concrete above.
PLACING SLAB CONCRETE FROM BUGGIES
Chute
Baffle
Chute
CORRECT
Place baffle and drop at end of chute so that separation is avoided and concrete remains on slope.
INCORRECT
To discharge concrete from a free end chute on a slope to be paved. Rock is separated and goes to bottom of slope. Velocity tends to carry concrete down slope.
WHEN CONCRETE MUST BE PLACED IN A SLOPING LIFT
CORRECT
Vertical penetration of vibrator a few inches into previous lift (which should not yet be rigid) at systematic regular intervals found to give adequate consolidation.
INCORRECT
Haphazard random penetration of the vibrator at all angles and spacings without sufficient depth to assure monolithic combination of the two layers.
For placing unformed concrete on slopes slipform screed should be steel faced, weighted and unvibrated. Concrete should be vibrated ahead of slipform.
Direction of travel
Unvibrated concrete
Immersion type vibrator
Steel plate
2"x4" Side form
Concrete for weight
No shoes at riding ends as form rides high when gravel gets under shoes. Keep riding edges sharp.
Vibrated concrete surface
PLACING CONCRETE ON A SLOPING SURFACE
SYSTEMATIC VIBRATION OF EACH NEW LIFT
CORRECT
Shovel rocks from rock pocket onto softer, amply sanded area and tramp or vibrate.
INCORRECT
Attempting to correct rock pocket by shoveling mortar and soft concrete on it.
TREATMENT OF ROCK POCKET WHEN PLACING CONCRETE
Direction of spreading
CORRECT
Turn bucket so that separated rock falls on concrete where it may be readily worked into mass.
Direction of spreading
Rock pockets form at bottom of slab
INCORRECT
Dumping so that free rock rolls out on forms or subgrade.
IF SEPARATION HAS NOT BEEN ELIMINATED IN FILLING PLACING BUCKETS
(A TEMPORARY EXPEDIENT UNTIL CORRECTION IS MADE)

Fig. 5—Correct and incorrect placing methods affecting consolidation

11. Where vibration is used to full advantage for consolidation of newly placed concrete, no supplemental spading or other working of the concrete is necessary. At corners, obstructions, block outs, congested reinforcing, under windows, and elsewhere where good consolidation may be in question, excellent results will be obtained by carefully attending to these places with ample, properly applied additional vibration. In unusually difficult situations supplemental form vibration, or form vibrators attached to reinforcing, at these points will secure good workmanship. Where air bubble holes are objectionable on a vertical surface, experience has shown they will be largely eliminated by using up to 100 percent more vibration than is necessary merely to insure solid filling. If this extra vibration appears to overvibrate the concrete, less water should be used in the concrete mix. Neither extra vibration nor spading will remove air bubble holes from surfaces molded under sloping forms; only such treatments as vacuum forms or absorptive form lining will remove air bubble holes from such surfaces.

Provision for handling concrete of the proper consistency

12. Objection is frequently made on the job to a relatively stiff consistency which will not flow down a certain chute, drop out of a certain bucket or hopper, or discharge through certain gates, although it is freely admitted that it is readily workable in place, particularly when properly vibrated. Such an objection is not valid and should not be sustained if the drier consistency has been determined in advance to be of practicable workability in the forms and has been made a requirement of the specifications. It is the function of concrete handling and placing equipment to handle and place concrete of proportions and consistencies that can be properly consolidated by vibration. Limitations on consistencies and proportions should not be imposed by inadequate chutes, hoppers, buckets, or gates.[1,4]

13. Thus it is important, in the design and approval of concrete handling equipment that chutes, where necessary, be amply steep, metal or metal lined, round bottomed, of large size, rigid, and protected from overflow. Discharge gates, bucket openings, and hoppers should be large enough to pass quickly and freely concrete of the lowest slump likely to be found practicable for placing in the forms by means of vibration. In many cases chutes steeper than 2 to 1 should be used, and double or triple the usual area of hopper outlets and gates would not be excessive.

14. Except where loss of slump due to delayed placing is unimportant and no increase in the original amount of mixing water is made because of it, or when remixing without additional water after a delay is desirable and practicable for reduction of shrinkage due to setting, every effort should be made to keep as short as possible the time elapsed from the moment water and cement come together until placement and consolidation of the concrete. This facilitates control of uniformity of consistency of concrete in the forms, and reduces to a minimum the water content of the concrete and variable loss of slump between mixer and forms. For this same reason concrete should not

be exposed in thin streams in long chutes or on long conveyor belts in which condition it is subject to the variable effects of weather. It should not be necessary to mix the concrete any wetter than it may be worked in the forms by means of vibration.

VI. GENERAL CONSIDERATIONS

1. All concerned with concrete work should remain aware of the importance of keeping its unit water content to a minimum.[1,4,7,12] Even though water-cement ratio is kept the same, an increase in unit water content increases shrinkage cracking, and as a result of the increased cracking, concrete may lose a measure of durability. An example is the case in which both cement and water are added without changing the water-cement ratio but with an increase in slump. Another is when increases in cement and water are required by increases in fines or by a finer grading of aggregate, but with little change in consistency. Where indiscriminate additions to water content increase the water-cement ratio, strength and durability are also adversely affected. The more a form is filled with the right combination of solids, and the less it is filled with water, the better will be the resulting concrete. To this end, moderation in the use of water, cement, and fine aggregate, together with aggregate graded to the largest practical maximum size, should be consistently practiced. Only as much cement should be used as is required to obtain adequate strength and other essential properties. Only as much water and fine aggregate should be used as is required to obtain concrete with no more workability than is needed for proper placing and consolidation by means of vibration.

Construction joints

2. For the sake of appearance, it is important that proper attention be paid to the mark made by a construction joint on exposed formed surfaces of concrete. Irregular construction joints should not be permitted; a straight line, preferably horizontal, should be required by filling forms to a grade strip (Reference 4, p. 244). Where a groove along a construction joint will not detract from appearance, this is an excellent method of obscuring the unsightliness which so often is associated with construction joints. Either a Vee or a beveled rectangular strip can be used as a grade strip to form a groove at the construction joint. Unless special precautions are taken to secure forms tightly against concrete at the joint, an unsightly offset is sure to occur and mortar from the next lift of concrete will leak down and disfigure concrete below the joint. Best assurance of form tightness is obtained when forms are reset and securely anchored with only 1 in. of form sheathing bearing on the concrete below the joint line made by the grade strip or groove strip (Reference 4, p. 244).

3. The surface of the construction joint should be prepared in a manner that will insure bonding with the concrete later placed on it, and if the joint is to be watertight, preparation of the joint must be particularly thorough.

It is not difficult to obtain a good joint where good quality low-slump concrete has been used at the top of the lift and has not been overworked. Where wet consistencies have been used, or where excessive vibrating or working has brought water or mortar to the surface, concrete at the surface of the lift is usually so inferior that it is not easy to obtain a good joint.

4. In the latter case the so-called green cut cleanup is a good expedient for such an inferior joint condition, which on good work would not be encountered. The first step should be the removal of all laitance and inferior surface concrete by means of a strong jet of air and water, at approximately 100 psi, after the concrete has hardened sufficiently to prevent the jet from raveling the surface below the desired depth and from forming cloudy pools of water that will leave a film on the surface when they dry. The surface of joints so treated should be moist cured especially well, preferably by a 1-in. layer of saturated sand, and, if possible, the surfaces should never be permitted to become dry during the interval before concrete is placed on it. Before placing new concrete the surface should be restored to the bright clean condition existing immediately following the green cutting by means of another washing with the air water jet, vigorous brushing with fine wire brooms, or by sandblasting as necessary. If the surface has been properly wet-sand cured, very little final scrubbing or sandblasting will be necessary.

5. Green cutting, followed by a thorough final cleanup can also be used with good results where quality of concrete at the joint surface is good. It produces no better results, however, than can be readily obtained, on surfaces which have been properly placed at the right consistency, by omitting any initial treatment and removing the surface film and dirt to a bright new surface by sandblasting and washing immediately prior to placing the new concrete. The green cutting method, including a comparable final cleanup, is usually found to be more expensive than the final sandblasting method and is not as foolproof. Final sandblasting without initial cleanup has been found to produce excellent results economically on horizontal joint surfaces of mass concrete that has been placed at a 2 in. slump or less with the aid of vibrators, if the surface is protected from excess working due to setting form anchors, the endeavor to embed all coarse aggregate, and the general job traffic until the concrete has hardened. Excessive working of surface concrete by job traffic is a common cause of inferior joints and a concerted effort to keep it to a minimum is worthwhile.

6. In all cases the new concrete should be preceded by about $\frac{1}{2}$ in. of soft mortar of the same proportions as that in the concrete. When accessible, this should be scrubbed into the surface of the joint with wire brooms. In column forms and deep narrow forms where $\frac{1}{2}$ in. of mortar may seem inadequate, it is preferable to follow the mortar with several inches of concrete containing mortar in excess of that in the usual mix and possibly containing coarse aggregate somewhat reduced in maximum size rather than use large quantities of straight mortar.

Finishing of unformed surfaces

7. For the most durable results in any finished, unformed concrete surface, the following procedures should be carefully followed. The concrete used should be of the lowest slump that can be properly consolidated. Preferably consolidation should be by means of vibration or some equally effective type of mechanical equipment. With such equipment or vibration, slump should be less than 2 in. Following consolidation, the operations of screeding, floating, and first troweling should be performed in such a manner that the concrete will be worked and manipulated *as little as possible* in obtaining the desired result. Overmanipulation brings inferior fines and water to the top and this impairs the quality of the finished surface; it is a direct cause of checking, crazing, and dusting. For the same reason each step in the finishing operation, from the first floating to the final floating or troweling, should be delayed *as long as possible* and yet permit the desired result to be obtained. Free water is unlikely to appear and accumulate between finishing operations if proper mix proportions and consistency are used. If water does accumulate it should be removed by blotting with mats, draining, or pulling off with a loop of hose, so the surface can lose its water sheen before the next finishing operation is performed. Under no circumstances should any finishing tool be used in an area before accumulated water has been removed. Neither should neat cement be worked into the surface to dry such areas. It is possible, however, to get good results from correctly performed mortar topping. Mortar topping should be placed on and worked into base concrete before the base concrete sets and the mortar consistency, consolidation, and finishing should be as described above. It is preferable, however, to place concrete of such proportions, quality, and consistency that the surface can be finished directly to the texture desired without excessive delay.

Cold weather concreting

8. For detailed discussion of good practice in winter concreting, reference is made to the recommendation of ACI Committee 604.[6] Briefly, this recommends that ordinary air-entrained concrete be placed and held at a minimum temperature of 50 F for 3 days, and that the concrete contain 1 percent calcium chloride by weight of cement when mean daily temperature is less than 40 F. When mean daily temperature is higher than 40 F for several days, calcium chloride is omitted and concrete temperature need be kept only above freezing for the first 48 hr. Proper insulation in close airtight contact with form sheathing or unformed concrete surfaces, has been found fully capable of maintaining recommended concrete temperatures in any weather not too cold for attendant construction operations. Steam released under a windproof tarpaulin enclosure is an excellent protection, provided concrete temperatures are kept moderate, since a moist atmosphere favorable to curing, as well as protection from freezing, is afforded. Concrete should not be overheated, either before or after placing; the opportunity should be taken in cold weather to mix, place, and cure concrete at comparatively low temperatures

as such concrete has highest strength and less cracking. Corners, edges, and surfaces of concrete are particularly vulnerable to freezing and the need for their adequate protection must be emphasized; in fact they may be frosted when the dry bulb temperature is above 32 F, if the wet bulb temperature reaches 32 F. As at any other time, regardless of the limited protection from freezing necessary for good durability, sufficient unimpaired curing time must be provided during and after periods of protection as is required to develop desired strength, and drying from overheating should be avoided.

9. With ample protection of corners, edges, and surfaces from freezing, the minimum temperature of mass concrete when placed may be as low as 40 F because heat is lost more slowly from such massive concrete.

Hot weather concreting

10. For best ultimate quality, concrete should be placed at the lowest practicable temperature at all times, especially in hot weather. Concrete temperatures can be kept down by concreting only at night, sprinkling or cooling the aggregate, sprinkling subgrades and surrounding areas, avoiding hot cement, and by using cold mixing water, possibly including ice. Some effects of high temperature can be offset by using an approved water-reducing retarder. Any combination or degree of these practices that may be feasible is desirable and should be required, including an upper limit of 75 F on temperature of concrete delivered by truck mixers or to be pumped by pipeline, unless an approved water-reducing retarder is used. Curing should preferably be obtained by sprinkling or covering with wet burlap for its additional cooling value, even if only for 24 hr prior to application of a sealing compound for curing. Only white-pigmented sealing compounds should be used for this type of curing because of their heat reflective value. Calcium chloride should not be used in warm weather. The recommendations of ACI Committee 605 for concreting in hot weather should be followed.[15]

Forms

11. Workmanship in placing concrete is largely judged by appearance of the work on removal of forms. Appearance is usually an important quality of a concrete job which may be greatly influenced at the time concrete is placed. It is, therefore, necessary that the following precautions be included in these recommendations. For further information on forms for concrete, reference is made to the work of ACI Committee 622, Formwork for concrete.[18,19]

12. With the general adoption of vibration for the consolidation of concrete, forms must not only be built substantially, but they must also be tight; otherwise, unsightly sand streaks and rock pockets will be caused by loss of mortar made unusually liquid during vibration. Immediately prior to placing concrete, thorough care should be exercised to insure that all form panel joints, corners, and connections, and all seams between all types of sheathing are tight enough to hold fluid mortar. Unsightly bulges and offsets at horizontal joints should be avoided by resetting forms with only 1 in. of sheathing overlapping the concrete below the line made by a grade strip and by securely

tying and bolting the forms close to the joint (Reference 4, p. 244). Formed grooves also obscure construction joints and improve appearance when they are well arranged (Reference 4, Fig. 109).

13. Form ties should be used which cause holes of minimum size in the form sheathing; if such holes are found to leak, they should be plugged effectively. Moreover, ends of form ties protruding from the concrete should be so designed and so removed that adjacent concrete will not be spalled, and a hole of minimum size will be left. Filling of these holes may not be necessary, but if it is, the work should be done in a manner that will secure sound, nonshrinking, inconspicuous fillings (Reference 1 and Chapter VII of Reference 4).

14. Forms should be protected from deterioration, weather, and shrinkage prior to concreting by proper oiling or by effective wetting. Form surfaces should be clean and of uniform texture. Where they are permissible, reused forms should be carefully cleaned and oiled. Steel forms should be thoroughly cleaned but never sandblasted nor abraded to bright metal. Where "peeling" is encountered with steel forms, leaving the cleaned, oiled, forms in the sun for a day, or vigorously rubbing the affected areas with liquid paraffin, or applying a thin coating of lacquer will usually improve the condition. Sometimes peeling is the result of abrasion of certain form areas from impact during placement, and improvement can be made by protecting these spots temporarily with plywood or metal sheets. It is generally less expensive and more satisfactory to obtain the desired surface effect by proper treatment and preparation of the forms than it is to obtain it by working over the concrete surface after forms are removed.

15. Forms should not be permitted to impair the quality of concrete filling them. Ample access should be provided for proper cleanup, placement, working and vibration of the concrete, and inspection of these operations, so that concrete of excessive slump or sand content, or smaller size rock, will not be proposed as an expedient for getting a presentable job.

16. Horizontally moving slip forms should not be used on slab work on which standard types of paving and finishing machines can be used to accomplish desired results. Such machines should be equipped with effective vibration and mechanical devices for working and consolidating the concrete. However, for slope paving, in preference to fixed forms, a continually moving, unvibrated, steel-faced slip form should be used (Fig. 5). The same should be used for concrete in steeply curved inverts. Concrete should be vibrated in front of the headboard of such slip forms.[5]

VII. CLOSURE

In general, each of the foregoing recommendations should be considered insofar as applicable in connection with each concrete job regardless of its size or circumstances. It should be noted, however, that although performance on this basis may be selected, it will not be obtained unless specifications are prepared accordingly and effective inspection is established with auththority to get such results.[1]

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Report by the International Council for Building Research

Load Factors

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This committee studied the various sources of error in assumptions, design, workmanship, or evaluation of materials, and arrived at some recommendations in a very philosophical manner. The same ideas and numerical values were adopted by the European Committee on Concrete in Rome, April 1957. The entire analysis is best read in the original report, but a few of the highlights are presented in this abstract.

SAFETY CALCULATIONS should fix a satisfactorily low upper limit to the probability of collapse and achieve a minimum value for the cost of construction, the maintenance cost during the useful life, the cost of a hypothetical insurance premium to cover accident risks, material losses, and possible loss of human life, with consideration to possible criticism of public opinion and similar factors of a psychological nature.

DESIGN DATA

Structural design proceeds from certain known data:

1. Applied forces, which represent a variety of types of loading

2. Geometric dimensions of the different parts of the structure, as given on the plans

3. The strength of the materials used, based on tests of specimens of those materials and on accumulated experience, and

4. Fundamental design assumptions or hypotheses.

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Errors in data

There are a number of sources of error in such data, some of which include:

1. Applied forces which exceed the original estimates

The probability of certain differences between actual applied forces and those assumed in the calculations is measured by the standard deviation, i.e., the root-mean-square or $\sqrt{\frac{\Sigma (x - \bar{x})^2}{n}}$, and will differ with the nature of the loads, being greater, for example, in the case of wind loads than for the pressure of still water in a reservoir.

2. An actual distribution of loads significantly different from those assumed in design

3. The degree of precision of the design calculations themselves

4. The accuracy of the basic design assumptions and hypotheses

The designer should take into account the various loading alternatives. The superposition of loadings of opposite sign may lead to very low resultant loading. The designer should not make any fundamental error, leading possibly to substantial increase of tensile stresses, or, even worse, to a change in the direction of the forces that might cause a tie member to buckle. In many instances, the most unfavorable loading can be obtained by superimposing the various forces (dead weight, live load, wind, thermal changes, seismic effects, etc.). If each of these forces attains its maximum value very rarely, the probability that their maxima will be coincident is very slight. In that case, the value of the load factor K_L (described later) should be reduced; certain specifications allow for a reduction of 20 percent.

5. Geometrical dimensions differing from the plans

6. The strength of the materials being less than that assumed in the calculations

7. Decrease in the strength of the materials with the passage of time

8. Variations in the quality of the workmanship

An error in the location of reinforcement may be more serious in the case of a thin slab than in a beam of large proportions. Hence, the load factor K_L might be higher in the former than in the latter instance.

An example in the variation of quality of material occurs in very long reinforced concrete structural units. Such variations are not always within the control of the contractor and should be covered by a reduction factor K_f (described later). The designer could specify a minimum "characteristic" strength f_{ck} for which the contractor is responsible. The contractor should test concrete under the same conditions of mixing, vibration, compaction, and by the same means as are applicable to the material employed in the structure. If these tests show large variations, increase the mean value of the strength f_m (e.g., increase the cement content). It is always possible to control, and maintain at the site, the minimum value of the characteristic strength.

9. Changes in the strictness of control over the work

10. The permissible probability of failure, which ought to vary inversely with the magnitude of the damage from possible failure

11. The behavior of materials at failure

Steel, being ductile, shows signs of distress, while failure in concrete may be a sudden collapse.

DESIGN PRACTICES

In the past, two methods have been employed to obtain adequate strength in spite of errors:

1. Instead of using the ultimate strength f of the materials (i.e., f_c' for concrete and f_y for steel), a reduced stress is used, called the "permissible" (or design) stress, obtained by dividing the ultimate stress by a suitable strength factor, and
2. The several loads are suitably increased for ultimate loads, i.e., ultimate strength design or limit design.

The committee considers that available statistical data, even though incomplete, are helpful for establishing usable values, and that it is advisable both to increase the assumed working load, along lines suggested by the theory of probabilities, and to reduce the assumed strength of the materials in a similar manner.

For a normal Laplace-Gauss distribution, the standard deviation σ obtained from a sufficiently large number of measurements, corresponds to a 16 percent probability of being either within the range $-\infty$ to $-\sigma$ or of $+\sigma$ to $+\infty$. Because these conditions may not be satisfied by the actual distribution of data, one can either multiply the factor $(1 \pm \sigma)$ by a correction factor F_1 to become $F_1 (1 \pm \sigma)$, or multiply the coefficient σ by a factor F to become $(1 \pm F\sigma)$, so that the characteristic values shall have a very small probability of being exceeded. If, for example, $F = 3$, the probability is 1.35×10^{-3}.

DETERMINATION OF LOAD FACTORS

The committee studied the three questions involved: (1) selection of the basic loads, (2) selection of the basic strengths (or elastic limit for steel), and (3) methods of calculation to insure that under action of the basic loads the basic strengths will not be exceeded.

The committee proposes that the standard deviation corresponding to $F = 1$ be used in connection with both correction factors, K_L being an increment of the assumed applied forces (loads) and K_f being a reduction in the assumed strengths (stresses).

Selection of basic loads

The basic ultimate applied loads for each type of building may be expressed:

$$L_b = L_m (1 \pm F_L\sigma) K_L$$

where

L_b = basic ultimate load

L_m = maximum load which, from a statistical investigation of a group of structures of similar type and life, has a 0.50 chance of being exceeded

σ = standard deviation, from statistical investigations of this kind of load

F_L and K_L = coefficients to insure the same probability of occurrence or nonoccurrence of any greater load once during the life of the structure

Since different factors may influence the different types of assumed loads, one can then introduce coefficients, K_{L1}, K_{L2}, K_{L3}, with values suitably determined by assessing each type of load, so the final result becomes:

$$L_b = K_{L1} L_{m1} (1 \pm F_1\sigma) + K_{L2} L_{m2} (1 \pm F_2\sigma) + K_{L3} L_{m3} (1 \pm F_3\sigma) \ldots\ldots\ldots\ldots$$

that is

$$L_b = \sum_{i=1}^{i=N} K_{Li} L_{mi} (1 \pm F_i\sigma)$$

Increasing F_L or K_L will increase L_b, and the probablity of overload is reduced. It is recommended that F_L be taken as 1.0 and K_L as 1.25, so we write:

$$L_b = K_L L_{ch}$$

where $L_{ch} = L_m (1 + \sigma)$ = the characteristic load, a name applied for convenience to a load $(1 + \sigma)$ times the load L_m, which has a 0.50 chance of being exceeded once in the life of the structure.

For a normal distribution and for an accepted probability of 16 percent, the characteristic value of the load, i.e., $L_{ch} = L_m (1 + \sigma)$, coincides fairly well with the *average of the upper half of the values obtained from an adequate number of actual observations of applied loads of a given type.* A plus sign is applicable to loads which are unfavorable to the stability of the structure, a minus sign to loads which help the stability. For a retaining wall, the horizontal thrust of the soil would be multiplied by $(1 + \sigma)$; the dead weight of the wall, which tends to maintain stability, would be multiplied by $(1 - \sigma)$. (This last σ will be very small because possible variations in estimating the weight of the wall are small.)

Selection of basic strengths

The basic strengths of concrete and steel (f_c' for concrete, f_y for steel) may be expressed:

$$f_b = \frac{f_m (1 - \sigma)}{K_f} = \frac{f_{ch}}{K_f}$$

where

f_b = the basic design strength to be assumed in the calculations

f_{ch} = the characteristic strength, given by $f_{ch} = f_m (1 - \sigma)$

f_m = the mean value of the strength, obtained from a series of tests on specimens

σ = the standard deviation in such series of tests

K_f = coefficient to reduce the probability that the actual strength on the job could be less than the basic value

The variation factor F (not included in writing these equations above) may again be taken as 1, where the number of tests is adequate, at least 20 in number, and where the variability is well known from previous tests of the same type. The committee proposes to take K_f for concrete (i.e., K_{fc}) as 1.5 to 1.6 applied to tests of standard 6 x 12 cylinders at 28 days, and K_f for steel (i.e., K_{fs}) as 1.15 or 1.20 of f_y. For the accepted probablity of 16 percent, the characteristic value of the strength, given by $f_{ch} = f_m (1 - \sigma)$, coincides substantially with the *arithmetic mean of the lower half of the test results, and so can be obtained from job cylinders in a very simple way.*

A structure should be so dimensioned that its various structural parts, when subjected to the basic loads assumed in calculation, can not be stressed beyond the assumed basic strengths of the materials, as defined above.

Recommended numerical values for reinforced concrete

The load factor K_L may be taken as 1.25 provided the basic assumptions have been strictly verified, the nature of connections and supports carefully established, all design and calculation work precisely carried out, the quality of workmanship and site control fully reliable, and the damages incidental to possible collapse are in the normal range (i.e., not excessively high in lives or money).

If these requirements are not met, the above value of $K_L = 1.25$ should be corrected as follows (the last two being superimposable on the others):

A*	All conditions met	basic $K_L = 1.25$
B*	Design and calculation of average accuracy and uncertainty	increase by 15 percent to $K_L = 1.44$
C*	Average workmanship	increase by 15 percent to $K_L = 1.44$
D*	Design and workmanship both average (1.15 × 1.15)	increase by 32 percent to $K_L = 1.65$
E*	Very serious risk of damage in the event of failure†	increase by 15 percent (see below)
F*	Very slight risk of damage in the event of failure	reduce by 15 percent (see below)

Thus K_L for Case E superimposed on Case A becomes 1.44; on Cases B or C, 1.65; and on Case D, 1.90. If Case F is combined with A, B, C, or D, then K_L becomes 1.06, 1.22, 1.22, or 1.41, respectively.

The strength factor for concrete K_{fc} can be between 1.50 and 1.60; minimum, 1.50.‡

The strength factor on the elastic limit of steel can be between 1.15 and 1.20; minimum 1.15.‡

*All based upon present European standards of performance.
†Accidents without warning signs and of exceptional severity, e.g., theatres, exhibition halls, public stands, dams built in highly-populated regions.
‡For an underreinforced member, use the strength factor K_{fs} depending on the yield point of the steel; for an overreinforced member, use the factor K_{fc} based upon the cylinder strength of concrete.

For live loads in dwellings, the committee accepts a mean value of L_m = 130 kg per sq m (27 psf). For an accepted probability of ± 17.7 percent, the characteristic value will be L_{ch} = 130 (1 + 0.177) = 153 kg per sq m (32 psf). This can be rounded off to 150 kg per sq m (31 psf), which coincides with the requirements of a large number of national specifications.

For other types of buildings, the committee recommends a statistical investigation to determine suitable loadings. The dead weight and live load for a floor should be added without any reductions.

Dead load and meteorological loads

For dead weight of floors in buildings, the committee suggests a coefficient of variation of ± 8 percent; the characteristic value being obtained by multiplying the most likely value of the dead weight by the factor 1.08 for normal conditions and 0.92 if the dead weight operates to reduce the required section of a member (e.g., bending moment at the center of a continuous beam where the weight of adjacent spans reduces the positive value).

In the combination of dead load and live load with a simultaneous meteorological load, a reduction to 80 percent of the total combined effect is allowed. (In the case of a terrace, accessible to the public, and designed for a live load of 400 kg per sq m (83 psf), it seems unnecessary to add the load due to a possible layer of snow.)

The addition of two meteorological loads whose simultaneous action is improbable or impossible need not be allowed for (e.g., wind *may* remove snow). If the addition of meteorological loads must be considered, then for each structural part the most unfavorable of the following three factors should be adopted: either (1) 80 percent of the characteristic value of the simultaneous loads due to wind and snow, or (2) the characteristic value of the wind alone, or (3) the characteristic value of the snow alone.

For columns supporting several stories, a reduction factor not greater than 40 percent may be applied to the sum of the maximum imposed loads for all the stories, because of the slight probability that the maximum imposed load will act simultaneously on all the floors. For columns of lower stories, the margin of safety should be increased, due to the magnitude of the damage in case of failure, but the above-mentioned reduction of maximum imposed load can be maintained.

Other types of loading (e.g., the effect of earthquakes) should be allowed for in a similar way.

Discussion of this paper should reach ACI headquarters in triplicate by Feb. 1, 1959, for publication in the June 1959 JOURNAL.

Title No. 55-37

TWENTY-FOUR TWO-SPAN CONTINUOUS BEAMS were tested to determine the manner and degree of moment and shear redistribution after yielding of the first critical section. Support displacement conditions were selected to provide severe redistribution requirements.

Manner and degree of moment and shear redistribution were essentially the same for all beams, with redistribution beginning with the start of steel yield at the first critical section and becoming practically complete upon initial yielding of the final critical section. Transverse ties designed to resist all the shear at plastic collapse provided adequate protection against diagonal tension failure with one exception.

Concentrated plastic rotation values indicate a sufficient capacity for the attainment of crushing moments at all critical sections for the steel ratios and support displacements of these tests. The amount available for redistribution may become limited by high steel ratios.

Moment and Shear Redistribution in Two-Span Continuous Reinforced Concrete Beams*

By GEORGE C. ERNST†

The principal object of this investigation was to determine the manner and degree of redistribution of internal moment and shear in two-span continuous reinforced concrete beams under extreme conditions of support displacement. Nineteen tests were conducted under symmetrical loading with a concentrated load at the midpoint of each span, and five tests with a concentrated load at the midpoint of one span only. Steel ratios of 0.008, 0.016, 0.027 with a nominal concrete strength of 5000 psi were used. The quantity and spacing of the transverse steel was designed to take the full shear at plastic collapse, at or below a stress of 50,000 psi. Of the beams tested under symmetrical loading, ten contained top and bottom bars running the full length of each beam, but in the other nine beams the top bars were cut off in accordance with the minimum requirements of ACI 318-56.

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In general, the supporting conditions were (a) no initial displacement of supports, (b) development of tensile yield in the bottom bars before contact with the center or an end support, and (c) development of tensile yield in the top bars before contact with the outer supports. Procedures (b) and (c) were considered extreme conditions of support displacement for internal moment and shear redistribution, with a high concentration of plastic hinging required at the load for (b) and at the midsupport for (c).

SPECIMENS AND TEST METHOD

Test specimens

All test specimens were 6 x 14½ in. in cross section, 19 ft long, with two spans of 8 ft 6 in. each. Three groups of five beams each were made with top and bottom bars extending throughout the full length of the beams, and are designated by the letter E. Three additional groups of three beams each were made with the top bars cut off in accordance with the minimum requirements of ACI 318-56, and are designated by the letter C. Each group contained a different amount and arrangement of steel, as shown in Table 1.

The individual beams are designated by a number referring to the bottom steel ratio and the letter E or C for extended or cut off top bars, followed by a number indicating one of the loading cases in Fig. 1. For example, beam 16E3 indicates a bottom steel ratio of 0.016, top bars extended full length, and loading Case 3. All E-beams contained the same top steel ratio as in the bottom, but all C-beams contained two #9 bars as top reinforcement.

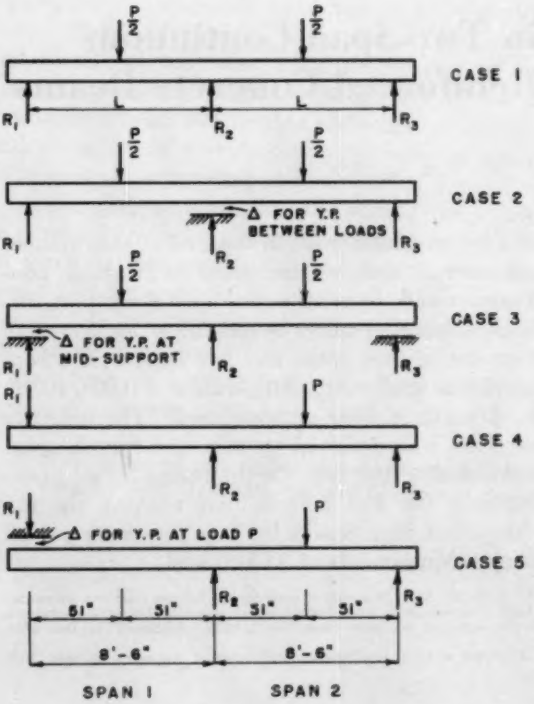

Fig. 1—Loading cases. Note: elevations of reactions not maintained after start of loading

TABLE 1—PRINCIPAL PROPERTIES OF BEAMS

d = 12.5 in., d' = 10.5 in., b = 6 in., t = 14.5 in., total length = 19 ft
Span 1 = Span 2 = 8.5 ft, average f_c' = 5280 psi

Group No.	Quantity of beams	Steel ratios		Transverse ties		Longitudinal bars	
		At loads p/p'	At R_2 p/p'	Outer ends to loads	Between loads	Bottom	Top
						Size and length	Size and length
8E	5	0.008/0.008	0.008/0.008	#3 @ 6 in.	#3 @ 6 in.	2 - #5 Full	2 - #5 Full
16E	5	0.016/0.016	0.016/0.016	#4 @ 6 in.	#4 @ 6 in.	2 - #7 Full	2 - #7 Full
27E	5	0.027/0.027	0.027/0.027	#4 @ 3 in.	#4 @ 3 in.	2 - #9 Full	2 - #9 Full
8C	3	0.008/0	0.027/0.008	#3 @ 6 in.	#4 @ 4 in.	2 - #5 Full	2 - #9 7 ft 8 in.
16C	3	0.016/0	0.027/0.016	#3 @ 6 in.	#4 @ 4 in.	2 - #7 Full	2 - #9 7 ft 8 in.
27C	3	0.027/0	0.027/0.027	#4 @ 6 in.	#4 @ 3 in.	2 - #9 Full	2 - #9 7 ft 8 in.

TABLE 2—PROPERTIES OF REINFORCING STEEL

Bar size	No. tests	Yield point stress, psi			Ultimate strength, psi			Beam group No.
		Minimum	Average	Maximum	Minimum	Average	Maximum	
#5	9	40,900	42,400	43,500	73,900	76,200	78,800	8E, 8C
#7	9	40,800	42,600	43,900	76,000	77,500	79,200	16E, 16C
#9	7	36,800	38,300	40,200	69,500	74,300	78,600	27E, 8C, 16C, 27C

The stirrups were made in the form of column ties of a size and spacing to resist the full shear at plastic collapse at not to exceed 50,000 psi.

The principal properties of all beams are listed in Table 1, including bar sizes and lengths as well as steel ratios.

Materials, fabrication, and curing

The test beams were precast in seven lots by a commercial precasting firm. The average strengths of three 6 x 12-in. cylinders for each lot, at time of testing the beams were

Lot 1 = 5550 psi Lot 4 = 5470 psi Lot 6 = 4680 psi
Lot 2 = 5320 psi Lot 5 = 5470 psi Lot 7 = 5330 psi
Lot 3 = 4970 psi Average of all cylinders = 5280 psi

Intermediate grade bars were used throughout. Typical stress-strain curves are provided in Fig. 2, and the minimum, average, and maximum strengths from all tension tests are listed in Table 2.

Test set-up and procedure

All beams were tested in a loading frame with a manually pumped 30-ton hydraulic jack at each load point. The load applied by each jack and the reaction at each end of a beam were measured by means of SR-4 load cells. Fig. 3 illustrates the general arrangement.

Steel strains were measured at the critical sections by SR-4 electric resistance strain gages placed on the bars through core holes after curing was completed. Gages for steel strain were also positioned on beams 16E1a, 16E3, and 27E1 such that the extent of yield spread might be determined.

Concrete strains were obtained from SR-4 resistance gages attached to the top surface 1 in. from each edge of each load bearing plate, and at the same location on the under surface at the midreaction bearing plate (Fig. 3).

Vertical displacements were obtained for all beams at each reaction, midpoint of each span, and at the quarter points of one loaded span. All displacements (deflections) were read to a thousandth of an inch, by means of an optical micrometer attached to a transit and an optical tooling scale at each displacement point.

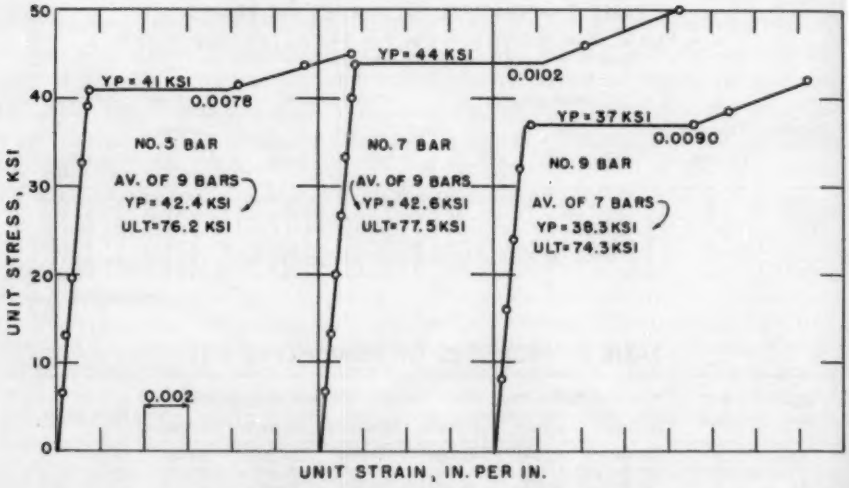

Fig. 2—Stress-strain curves for reinforcing bars

The test procedure consisted of taking incremental readings of load, strain, and deflection until no further load increase was possible by manual pumping. Such readings were also taken at yielding of the steel and at crushing of the concrete. Special requirements for each loading case of Fig. 1 are:

Case 1

a. The center support was initially placed at a level producing zero moment at the mid-reaction.

Case 2

a. Yield was developed in the bottom bars between the load points, without the center support.

b. The center support was next inserted and the loading continued.

Case 3

a. Yield was developed in the top bars without end supports.

b. The end supports were next inserted and the loading continued.

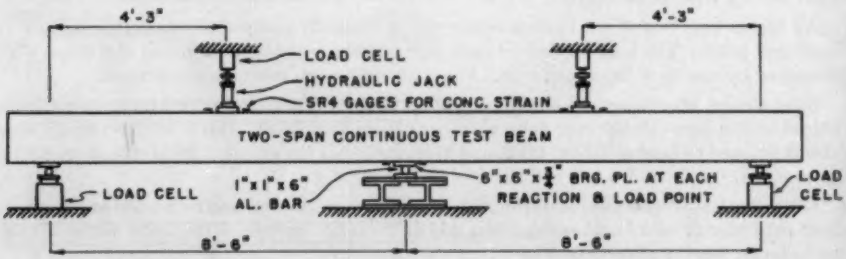

Fig. 3—Basic test set-up

Case 4

a. All supports were in contact at the beginning of load application.

Case 5

a. Yield was developed in the bottom bars at the load, without the end support on the unloaded span.

b. The end support on the unloaded span was next inserted and the loading continued.

After establishing the position of the supports at the beginning of each test, no attempt was made to adjust for changes in elevation of the supports caused by the deflection of the loading frame and fittings under load. Vertical deflections at the supports and load points were recorded for all beams, and the resulting relative deflections were tabulated (Tables 5 and 6).

TEST RESULTS

Principal test data

The principal test data are summarized in Tables 3 to 6. The shears, moments, and total load at yielding of the steel at the last critical section are given in Table 3 for all beams except 27C3. The latter beam failed in diagonal tension between the end of the cut off bars and a load point prior to insertion of the end supports, although neither 8C3 and 16C3 with cut off bars developed such a fracture. Table 4 provides the shears, moments, and total loads at the maximum applied load. The calculated loads of Tables 3 and 4 were obtained on the basis discussed in the section on analytical studies.

TABLE 3—SHEARS, MOMENTS, AND LOADS AT YIELDING* OF STEEL AT THE LAST CRITICAL SECTION

Beam No.	Loading case, No.	Shear, kips		Moment, in.-kips		Total load on two spans, kips		
		Outer	Inner	Positive	Negative	Test	Calculated	Test/Calc.
8E1	1	6.08	12.67	310	344	37.5	32.82†	1.14
8E2	2	6.45	13.00	330	334	38.9	36.82‡	1.06
8E3	3	6.51	13.54	332	359	40.1	34.20§	1.17
8E4	4	7.08	12.82	360	293	19.9	17.69§	1.13
8E5	5	7.52	12.38	384	248	19.9	18.81‡	1.06
8C1	1	7.01	21.60	358	747	57.4	52.62†	1.09
8C2	2	8.07	22.71	411	746	61.6	59.98‡	1.03
8C3	3	6.50	24.95	331	940	62.9	57.22‡	1.10
16E1	1	11.13	23.41	567	625	69.1	66.90†	1.03
16E1a	1	11.32	24.38	577	665	71.4	66.90†	1.07
16E2	2	13.31	25.69	679	630	78.0	71.90‡	1.09
16E3	3	11.30	22.95	575	594	68.5	70.50§	0.97
16E5	5	14.71	25.09	750	529	39.8	37.14§	1.07
16C1	1	11.59	30.01	590	940	83.2	77.42†	1.08
16C2	2	13.55	28.65	690	769	84.4	85.50‡	0.99
16C3	3	12.07	30.23	615	925	84.6	80.36‡	1.05
27E1	1	16.80	34.24	856	889	102.1	100.20†	1.02
27E2	2	19.95	34.95	1017	764	109.8	102.72‡	1.07
27E3	3	16.89	33.96	860	870	101.7	102.2 ‡	0.99
27E4	4	19.20	35.30	978	820	54.5	54.62§	1.00
27E5	5	19.70	35.50	1007	805	55.2	54.62‡	1.01
27C1	1	17.55	33.55	895	815	102.2	99.80†	1.02
27C2	2	Steel yield not developed at last critical section. See Table 4 for maximum values attained. Ratio of test to calculated load = 1.01.						
27C3	3	Steel yield not developed due to diagonal tension failure near load point 2, prior to insertion of end supports at R_1 and R_3.						

*Yielding as indicated by attainment of yield strain on SR-4 gages.

†M_p assumed at all yielding sections.

‡M_u at last critical section: $M = 0.9A_s f_s d$ at first critcal section, with f_s from stress-strain curve and test value of strain.

§Same as ‡, except that steel strain was taken as 0.015 due to inoperative test gages.

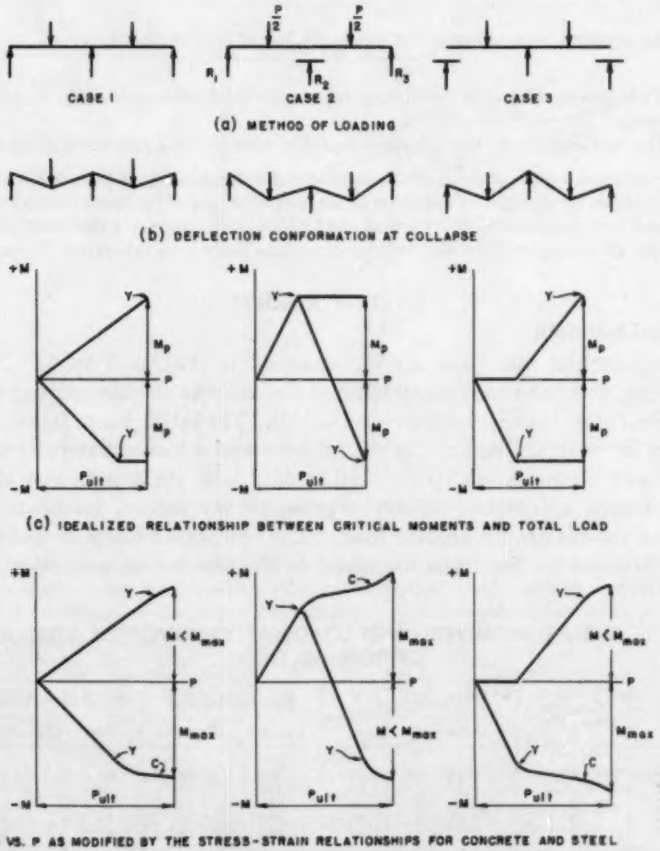
CASE 1
CASE 2
CASE 3
(a) METHOD OF LOADING
(b) DEFLECTION CONFORMATION AT COLLAPSE
(c) IDEALIZED RELATIONSHIP BETWEEN CRITICAL MOMENTS AND TOTAL LOAD
(d) M VS. P AS MODIFIED BY THE STRESS-STRAIN RELATIONSHIPS FOR CONCRETE AND STEEL

Fig. 4—Test behavior for $p = p'$

Deflections that were developed at initial yielding of the steel in the final critical section are given in Table 5. Deflections and concentrated plastic rotations developed by the maximum applied load are listed in Table 6. The deflections are referred to those supports for which the relative movements would be typical of the loading condition, with downward deflection as positive and upward deflection as negative.

Behavior under load

If the top and bottom steel ratios are equal at all sections, and are less than the balanced ratio,[1] the general behavior of the beams for loading cases 1, 2, and 3 should conform to that illustrated in Fig. 4. The loading cases are shown at the top, with the resulting collapse mechanism just below. The relationship between the critical moments and total load, based upon the assumption of idealized properties,[2,3] is sketched in Fig. 4c. However,

the lack of distinctly separated elastic and plastic properties in the concrete, tensile cracking, local bond conditions, etc., will modify the curves to something like those given in Fig. 4d.

The ratio of positive to negative elastic moments for Case 1 is 0.83, thereby requiring a relatively small amount of concentrated plastic rotation at the midsupport section in order to develop yield at the loads. Cases 2 and 3 are devised to produce large concentrated plastic rotations at midsupport and at the loads, respectively, before yielding of the steel at the last critical section. This larger amount of concentrated plastic rotation is represented by the greater distance between Y (yield of steel) and C (concrete crushing) in the sketches for Cases 2 and 3 in Fig. 4d.

Moment redistribution

Redistribution of internal resisting moments is initiated in a statically indeterminate structure by plastic rotation at the first critical section,[3,4] as illustrated in Fig. 4d. The degree of moment redistribution may be made more evident by superimposing the three curves of Fig. 4d, for which convergence should start at initial yielding of the first critical section. If the plastic properties of the material permit sufficient plastic rotation at the first critical section in each case, the moments should converge to a common value although not necessarily the same value for positive and negative moments.

TABLE 4—SHEARS, MOMENTS, AND LOADS AT MAXIMUM* APPLIED LOAD

Beam No.	Loading case, No.	Shear, kips		Moment, in.-kips		Total load on beam, kips			Remarks*
		Outer	Inner	Positive	Negative	Test	Calculated†	Test/Calculated	
8E1	1	8.37	19.73	427	580	56.2	42.1	1.33	Ultimate load
8E2	2	8.16	16.94	416	447	50.2	42.1	1.19	
8E3	3	8.85	18.15	451	474	54.0	42.1	1.28	
8E4	4	8.80	17.10	449	424	25.9	21.1	1.23	
8E5	5	8.92	15.58	456	344	24.5	21.1	1.16	
8C1	1	8.20	28.80	419	1050	74.0	63.7	1.16	Ultimate load
8C2	2	8.94	26.76	456	909	71.4	63.7	1.12	
8C3	3	7.81	29.19	398	1088	74.0	63.7	1.16	Ultimate load
16E1	1	14.97	32.83	762	910	95.6	77.3	1.24	Ultimate load
16E1a	1	14.34	31.66	731	882	92.0	77.3	1.19	Ultimate load
16E2	2	15.05	29.65	766	744	89.4	77.3	1.16	
16E3	3	14.91	30.99	760	818	91.8	77.3	1.19	
16E5	5	16.00	28.80	815	653	44.8	38.7	1.16	Ultimate load
16C1	1	14.32	38.33	730	1225	105.3	81.0	1.30	Ultimate load
16C2	2	14.69	31.61	747	862	92.6	81.0	1.14	
16C3	3	13.76	34.74	700	1068	97.0	81.0	1.20	
27E1	1	18.09	36.91	921	959	110.0	107.2	1.03	
27E2	2	19.95	34.95	1017	764	109.8	107.2	1.02	
27E3	3	17.58	36.07	895	941	107.3	107.2	1.00	
27E4	4	20.10	36.70	1023	846	56.8	53.6	1.06	Ultimate load
27E5	5	19.92	36.18	1015	828	56.1	53.6	1.05	Ultimate load
27C1	1	18.04	36.26	920	928	108.6	103.0	1.05	
27C2	2	20.81	32.69	1060	605	107.0	103.0	1.04	No yield at last critical section
27C3	3	Maximum total load = 21.2 kips. developed at diagonal tension failure at load point 2 prior to insertion of end supports at R_1 and R_3.							

*Ultimate capacity was considered to have been attained only for those beams having a reduction in load upon rapid hand pumping of the hydraulic loading jacks. In other cases neither a drop nor a further increase in load was produced by rapid hand pumping.

†All calculated loads were based on the assumption that the crushing moment had developed at all critical sections. M_c was computed from Reference 8 using average f_c' and Fig. 2.

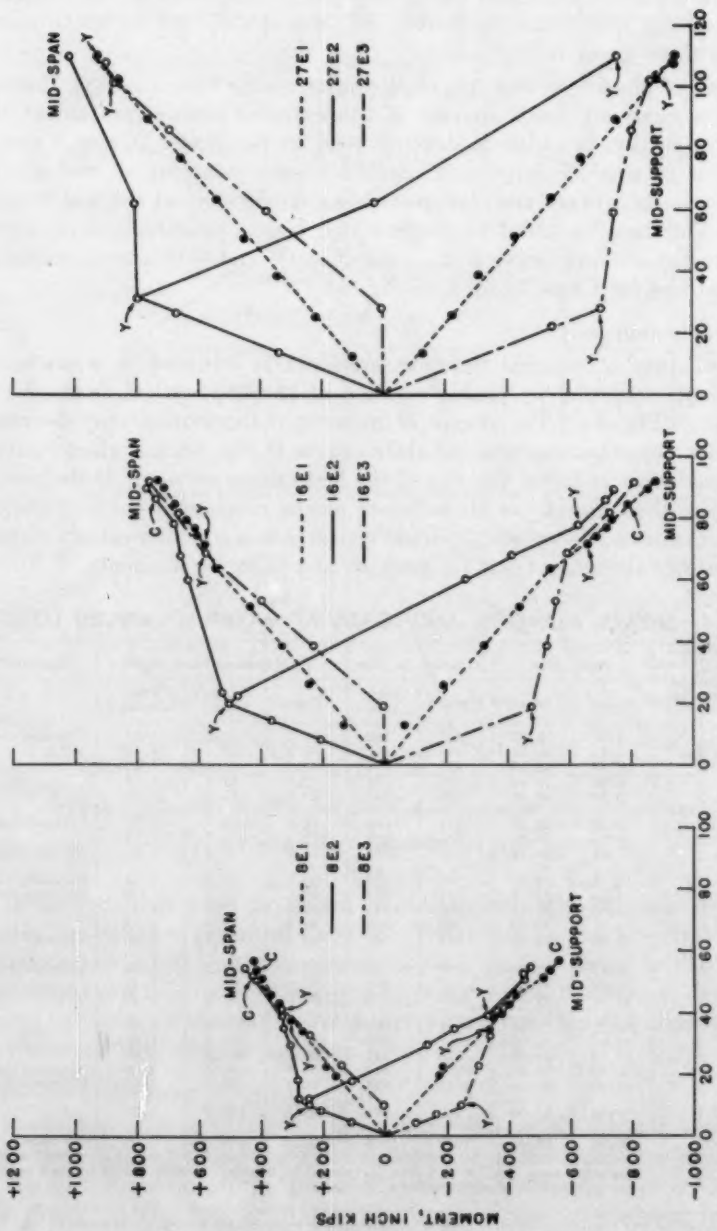

Fig. 5—Redistribution of moments for E-series, symmetrical loading cases

TABLE 5—DEFLECTIONS AT INITIAL YIELDING OF STEEL AT FINAL CRITICAL SECTION

Beam No.	Loading case, No.	Deflection,* in., at indicated location				
		Load 1	Load 2	R_1	R_2	R_3
8E1	1	0.151	0.132	0	0.014	0
8E2	2	1.155	1.150	0	1.157	0
8E3	3	0.375	0.383	0.518	0	0.518
8E4	4	—	0.444	−0.212	0	0
8E5	5	—	0.485	−0.396	0	0
8C1	1	0.317	0.325	0	0.197	0
8C2	2	1.816	1.668	0	1.713	0
8C3	3	0.575	0.404	0.397	0	0.397
16E1	1	0.168	0.154	0.041	0	0
16E1a	1	0.235	0.180	0	0.045	0
16E2	2	1.189	1.305	0	1.145	0
16E3	3	0.308	0.318	0.278	0	0.278
16E5	5	—	1.079	−0.962	0	0
16C1	1	0.167	0.216	0	0.035	0
16C2	2	1.789	1.536	0	1.628	0
16C3	3	0.532	0.573	0.633	0	0.633
27E1	1	0.306	0.351	0	0.153	0
27E2	2	1.571	1.481	0	1.529	0
27E3	3	0.327	0.346	0.366	0	0.366
27E4	4	—	0.951	−0.485	0	0
27E5	5	—	0.935	−0.814	0	0
27C1	1	0.442	0.428	0	0.289	0
27C2	2	See note in Table 3				
27C3	3	See note in Table 3				

*Positive deflections are down, negative deflections are up.

Such a convergence toward common moments is evident in the typical experimental curves of Fig. 5. The start of steel yield, as indicated by yield strain values from the SR-4 gages on the steel at the critical sections, is indicated by *Y*. Concrete crushing, when detected either visually or by a recession of strain through SR-4 gages on the concrete at the critical sections, is indicated by *C*. It is evident from these curves that initial steel yield at the last critical section denotes the point at which the redistribution is essentially complete. This is clearly shown by the shear redistribution curves of Fig. 6, discussed in the next section.

Shear redistribution

Fig. 6 shows the typical redistribution of shear by plotting the ratio of outer reaction divided by load on span as the abscissa and the ratio of applied load to load at initial steel yield at the last critical section as the ordinate. In this manner, the similarity of convergence for the three steel ratios of these specimens is clear. Also, the closeness of convergence to a common value at and beyond steel yield at the last critical section is shown.

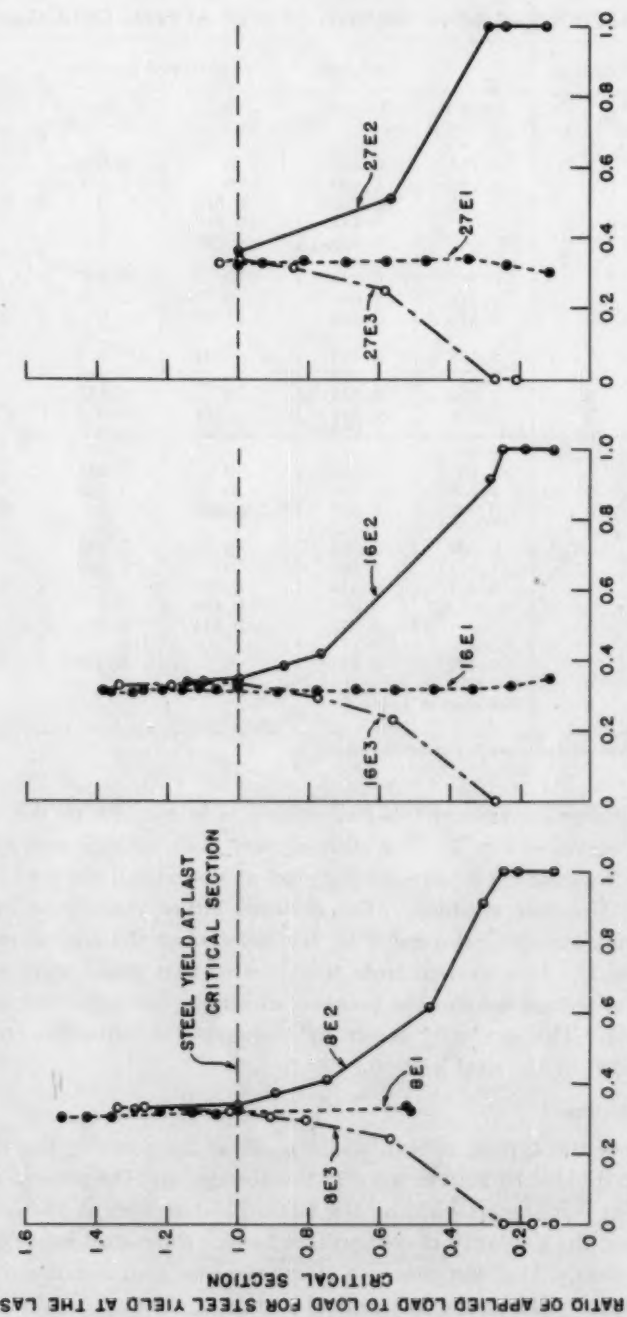

Fig. 6—Redistribution of shear for E-series, symmetrical loading

Attention is called to the fact that the diagonal tension steel was designed to resist more than the maximum shears resulting from the redistribution that would exist at plastic collapse. It was not an objective of these tests to determine the extent to which the customary design of shear reinforcement would provide protection against diagonal tension failure in the plastic range of loading.

Yield spread and crushing

The gages positioned for the purpose of determining yield spread showed that steel yield extended at least 17 in. from the center of load application just prior to the ultimate load for Beams 16E1a and 16E3 (steel ratios of 0.016 at top and bottom), but did not reach 10 in. for 27E3 (steel ratios of 0.027 at top and bottom). Considering that the edge of the bearing plate was 3 in. from the center of load application, these indications of yield spread are quite consistent with more detailed information from other tests.[1]

Because of the number of gage readings taken at each load increment, it was difficult to obtain the exact load at which crushing became evident visually or by a recession of strain in the vicinity of the loads and center support. The points marked C in Fig. 5 are those at which concrete crushing was clearly evident by either strain recession or visual inspection of the whitewashed surface.

NOTATION

L = Span 1 or Span 2, in.
M = moment, in.-lb
M_y = moment at start of steel yield, in.-lb
M_c = moment at crushing of concrete at $L/2$, in.-lb
M_c' = moment at crushing of concrete at mid-support, in.-lb
P = total load (sum of loads on both spans), lb
P_y = load calculated on assumption of M_y at all critical sections, lb
P_c = load calculated on assumption of M_c at all critical sections, lb
b = width of beam, in.
d = effective depth, in.
d' = distance between tension and compression steel, in.
t = total depth, in.
f_c' = ultimate concrete cylinder strength, psi
f_s = steel stress, psi
f_y = yield point of steel, psi
R = reactions, lb (with subscripts 1, 2, or 3 to indicate position)
A_s = area of tension steel, sq in.
A_s' = area of compression steel, sq in.
p = A_s/bd
p' = A_s'/bd
Δ_R = deflection of support, in.
Δ_1 = deflection at $L/2$ of loaded span, in.
Θ_p = concentrated plastic rotation, radians
ϕ_e = M_c/E_cI, in which E_c = concrete modulus and I = moment of inertia of transformed section in concrete
ϕ_c = unit rotation accompanying M_c radians per in.
ϕ_p = $\phi_c - \phi_e$
q = M_c/M_c' = ratio of positive to negative crushing moments

ANALYTICAL STUDIES

Calculated versus test loads

The moment at the first critical section may be substantially greater than the yield moment M_y when the steel just reaches its yield point at the last critical section. M_y may be computed by the customary assumptions of

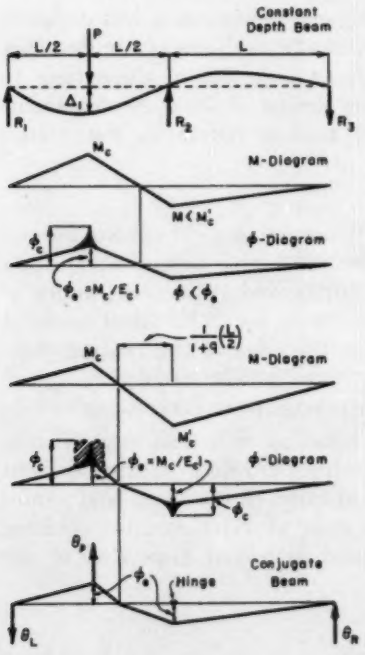

Fig. 7—Development of Θ_p in two-span continuous beam with load at center of one span only

linear distribution of strain, proportionality of stress to strain, and no tension in the concrete.[1] The general plastic moment equation[1] should be used if M_y is exceeded, but sufficient experimental information must be available for the determination of certain constants.[7] Such experimental data were not obtained for these tests, but a close approximation may be computed from $M = A_s f_s(0.9d)$ if f_s is determined from the actual strain measurements and the complete stress-strain curve for the steel. The calculated loads of Table 3 were based upon $M = A_s f_s(0.9d)$ at the first critical section and M_y at the last critical section, for all beams tested under Cases 2, 3, 4, and 5. M_y was used at all critical sections of Case 1 beams because of the almost simultaneous formation of steel yield at load and midsupport sections. For the beams of Cases 2, 3, 4, and 5, steel strains were in the work-hardening range when steel yield developed at the last critical section. The ratios of test load to calculated yield load listed in Table 3 range from 0.97 to 1.17, with only three values below 1.00.

The crushing moment was assumed at all critical sections in obtaining the calculated moments in Table 4. The average ultimate concrete strength and the steel stress-strain curves of Fig. 2 were used with Reference 7 to compute the crushing moments of the various sections. The ratios of maximum applied test load to calculated crushing load listed in Table 4 range from 1.00 to 1.33, with those for the steel ratio of 0.027 very close to 1.00.

Concentrated plastic rotations

The magnitude of the concentrated plastic rotation necessary for the attainment of the crushing moment at all critical sections may be calculated from a consideration of the lack of balance or closure of the M_c/E_cI diagram.[2] For example, if the load P on the two-span continuous beam of Fig. 7a is just large enough to develop the crushing moment M_c at the first critical section, the moment diagram will be that shown in Fig. 7b. The accumulated plastic angle change of the solid black area in Fig. 7c will slightly increase the negative moment above that for purely elastic conditions, by means of a

small amount of moment redistribution. Although up to this load the dominant action of the beam has been elastic, further increases will cause spreading of ϕ_c in the vicinity of the load along with a flexural adjustment until the crushing moment develops at the center support as shown in Fig. 7e. Inasmuch as the moment of resistance of a section is not greatly increased by strains greater than crushing,[1] the load that produces M_c at both critical sections for this two-span beam should be close to the ultimate load.

The accumulated plastic angle change in the vicinity of the load in Fig. 7e is represented by the concentrated plastic rotation Θ_p on the idealized conjugate beam of Fig. 7f, in which the small amount of plastic rotation at R_2 has been neglected. This conjugate beam may be used to obtain the magnitude of Θ_p necessary for the formation of the moment diagram of Fig. 7d. The following expressions for Θ_p and Δ_1 for the various loading conditions of these tests were developed by such an analysis in each case.

Case 1 or 2 with a downward movement Δ_R of the mid-support:

$$\Theta_p = \pm \frac{\phi_c L}{12(1+q)^2}(6q^2+q-3) \mp 2\frac{\Delta_R}{L} \begin{cases} + - \text{ if } \Theta_p \text{ occurs at } R_2 \\ - + \text{ if } \Theta_p \text{ occurs at } L/2 \end{cases}$$

$$\Delta_1 = \begin{cases} \dfrac{\phi_c L^2}{48(1+q)^2}(4q^3+7q+1) + \dfrac{\Delta_R}{2} \text{ if } \Theta_p \text{ occurs at } R_2 \\ \dfrac{\phi_c L^2}{24(1+q)^2}(2+3q-q^2) + \Delta_R \text{ if } \Theta_p \text{ occurs at } L/2 \end{cases}$$

Case 1 or 3 with a downward movement Δ_R of each outer support:

$$\Theta_p = \pm \frac{\phi_c L}{12(1+q)^2}(6q^2+q-3) \pm 2\frac{\Delta_R}{L} \begin{cases} + + \text{ if } \Theta_p \text{ occurs at } R_2 \\ - - \text{ if } \Theta_p \text{ occurs at } L/2 \end{cases}$$

$$\Delta_1 = \begin{cases} \dfrac{\phi_c L^2}{48(1+q)^2}(4q^2+7q+1) + \dfrac{\Delta_R}{2} \text{ if } \Theta_p \text{ occurs at } R_2 \\ \dfrac{\phi_c L^2}{24(1+q)^2}(2+3q-q^2) \qquad \text{if } \Theta_p \text{ occurs at } L/2 \end{cases}$$

Case 4 or 5 with upward movement Δ_R at outer support of unloaded span:

$$\Theta_p = \frac{\phi_c L}{12(1+q)^2}(2q^2+15q+11) + \frac{2\Delta_R}{L}$$

$$\Delta_1 = \frac{\phi_c L^2}{24(1+q)^2}(3q^2+11q+6) + \frac{\Delta_R}{2}$$

Deflections at $L/2$ and concentrated plastic rotations computed for these tests by use of measured support deflections substituted in the appropriate equations above are listed in Columns 7 and 8 of Table 6. For these tests,

TABLE 6—DEFLECTIONS AND CONCENTRATED PLASTIC ROTATIONS FOR MAXIMUM* APPLIED LOAD

Beam No.	Deflections† in in. at					Calculated values of			θ_p for P_{max} by test			$\frac{P_{max}}{P_c}$
	Load 1	Load 2	R_1	R_2	R_3	Δ_1 for P_c	θ_p for P_c	Relative severity of redistribution‡	Load 1	Load 2	at R_2	
8E1	1.913	1.464	0.025	0	0.025	0.116	0.00185 at R_2	1.4	0.0704	0.0528	0.0635	1.33
8E2	1.688	2.173	0	1.150	0	1.220	0.02119 at P	15.6	0.0396	0.0488	0.0230	1.19
8E3	1.333	2.007	0.502	0	0.502	0.355	0.01121 at R_1	8.3	0.0383	0.0649	0.0628	1.28
8E4	—	2.689	−0.211	0	0	0.454	0.01366 at P	1.4	—	0.1013	0.0438	1.23
8E5	—	1.123	−0.447	0	0	0.572	0.01828 at P	1.9	—	0.0399	0.0108	1.16
8C1	1.086	1.335	0	0.336	0	0.441	0.00754 at P	7.9	0.0328	0.0426	0.0302	1.16
8C2	2.163	1.950	0	1.749	0	1.854	0.03523 at P	37.0	0.0474	0.0390	0.0008	1.12
8C3	0.925	1.618	0.397	0	0.397	0.280	0.00684 at R_3	7.2	0.0261	0.0133	0.0193	1.16
16E1	1.500	1.350	0	0	0	0.104	0.00136 at R_3	1.0	0.0547	0.0489	0.0532	1.24
16E1a	1.234	1.196	0	0.025	0	0.117	0.00087 at R_2	0.6	0.0438	0.0422	0.0439	1.19
16E2	1.639	1.716	0	1.144	0	1.214	0.02109 at P	4.6	0.0378	0.0408	0.0182	1.16
16E3	1.012	1.060	0.252	0	0.252	0.230	0.00631 at R_3	15.5	0.0306	0.0326	0.0379	1.19
16E5	—	1.867	−0.994	0	0	0.846	0.02902 at P	3.1	—	0.0690	0.0200	1.16
16C1	1.012	1.113	0	0.012	0	0.101	0.00007 at R_3	0.2	0.0357	0.0397	0.0378	1.30
16C2	2.004	1.594	0	1.774	0	1.861	0.03449 at P	110.0	0.0400	0.0241	0.0000	1.14
16C3	0.874	1.620	0.614	0	0.614	0.401	0.01234 at R_3	39.3	0.0186	0.0478	0.0455	1.20
27E1	0.510	0.507	0	0.157	0	0.227	0.00172 at P	1.3	0.0132	0.0130	0.0114	1.03
27E2	1.571	1.381	0	1.529	0	1.599	0.02860 at P	21.1	0.0275	0.0201	0.0000	1.02
27E3	0.455	0.443	0.358	0	0.358	0.283	0.00838 at R_3	6.2	0.0067	0.0063	0.0149	1.00
27E4	—	0.938	−0.476	0	0	0.587	0.01886 at P	2.0	—	0.0327	0.0069	1.06
27E5	—	1.007	−0.815	0	0	0.756	0.02550 at P	2.7	—	0.0353	0.0049	1.05
27C1	0.494	0.515	0	0.321	0	0.395	0.00518 at P	4.6	0.0091	0.0099	0.0043	1.05
27C2	1.876	1.634	0	1.714	0	1.788	0.03250 at P	29.0	0.0359	0.0265	0.0000	1.04
27C3	See note in Table 4											

*See Table 4 and first note below table.
†Postive deflections are down and negative deflections are up.
‡Relative severity of redistribution required to attain M_c at all critical sections, based upon no support deflection as unity.

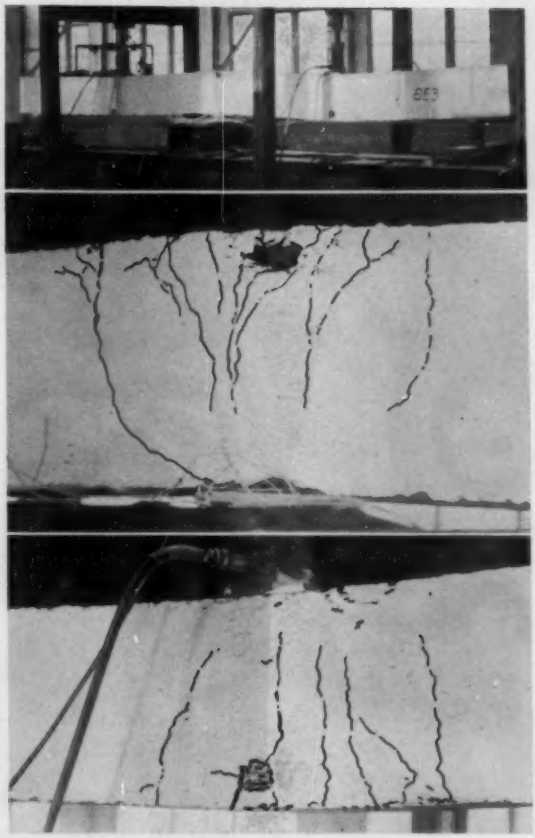
At maximum applied load
Cracking at first critical section at mid-support
Cracking at second critical section at load point

Fig. 8—Beam 8E3, deflection and cracking

beams with steel ratios less than 0.027 exhibited a capacity for concentrated plastic rotation in excess of that required in Column 8. This excess was utilized in raising the load above that for crushing at all critical sections and in adjusting to the various support displacements. Cracking of the critical sections at the maximum applied load was typical of that shown in Fig. 8. The development of cracking up to the maximum applied load was typical of previous tests on plastic hinging.[1]

SUMMARY

Twenty-four two-span continuous beams of reinforced concrete were tested to determine the manner and degree of redistribution of moment and shear for extreme conditions of support displacement. Three support conditions for symmetrical loading and two for unsymmetrical loading were investigated. Top bars were extended the full length in 15 beams, and were cut off in accordance with ACI 318-56 for the remaining beams.

The results from the entire investigation may be summarized as follows:

1. Redistribution of moment and shear was initiated by the beginning of steel yield at the first critical section, and was essentially complete at initial yielding of the steel at the last critical section. Increases in load beyond initial yielding of the steel at the last critical section were developed primarily by proportionate increases in resisting moments and shears, rather than by redistribution.

2. The manner of redistribution of moment and shear was the same for all beams, irrespective of the differences in steel ratios and length of top bars.

3. Design of the diagonal tension reinforcement to take all the shear at plastic collapse insured against diagonal tension failures, excepting in the case of 27C3 when subjected to negative bending near the ends of the cut-off bars. The fracture of 27C3 provides evidence that beams with cut off bars are susceptible to diagonal tension failure when subjected to negative bending near the ends of the cut off top bars.

4. The spread of steel yield, where observed, was consistent with previous tests.

5. The calculated load based upon crushing moments at all critical sections was less than the maximum applied experimental load, in all cases.

6. The assumption of M_y at all critical sections would provide an estimate for the lowest load at which redistribution of moment and shear would be complete.

7. Beams with steel ratios less than 0.027 exhibited a capacity for concentrated plastic rotation in excess of that required for redistribution of moments and shears up to the development of crushing moments at all critical sections.

ACKNOWLEDGMENTS

This project was conducted under a grant from the National Science Foundation for a study of reinforced concrete in the plastic range.

The tests were carried on in the Willow Run Laboratories of the University of Michigan Engineering Research Institute, through a cooperative agreement with the University of Nebraska. Prof. L. M. Legatski, Department of Civil Engineering, University of Michigan, was in charge of the project for the Engineering Research Institute. Successful completion of the tests during the short period of July and August of 1957 was primarily due to Professor Legatski's planning and direction in connection with the fabrication and storage of the test specimens, and the making of special equipment prior to July 1957. Appreciation is also due H. F. Poehle, assistant director, Engineering Research Institute, University of Michigan, for arranging the conditions of the agreement between the two universities.

The beams were precast by the Adams Concrete Products Co., Ypsilanti, Mich., in accordance with specifications supplied by the author. The successful functioning of the beams for research purposes is clear evidence of the high quality of the work.

Acknowledgment is given to Stephen C. Ernst for the care and consistency exhibited throughout all tests in the use of the optical tooling micrometer for deflection readings, and in the assistance given during preparation, installation, and removal of the beams.

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Discussion of this paper should reach ACI headquarters in triplicate by Feb. 1, 1959, for publication in the June 1959 JOURNAL.

Title No. 55-38 brings a report on the stress and strain conditions at the instant of cracking for small bars of cement mortar whose shrinkage was restrained from an early age by a centrally placed steel rod. Dr. Blakey shows that cracking probably starts as a definite elastic strain in the cement paste phase of the mortar. He also reports on the creep and shrinkage of mortars.

influence of water-cement ratio on mortar in which shrinkage is restrained

By F. A. BLAKEY*

Throughout the last half century or more much research has been done on the shrinkage of concrete and there is now a fairly clear understanding of the effect on this shrinkage of changes in any of the factors such as water-cement ratio and aggregate size that may be controlled during the manufacture of concrete.

However, until the recent development of prestressed concrete the deformation caused by shrinkage was seldom of practical importance; far more interest has been centered on the stresses set up and the consequent risk of cracking when the shrinkage is restrained. These stresses depend not only on the shrinkage of the concrete but also on its elastic and inelastic (creep) properties. Numerous studies of creep in concrete have been made so that the knowledge on this subject has now achieved much the same level as for shrinkage, but most of the experimental work has been done under constant stress conditions and the results are not necessarily applicable to the conditions of increasing or variable stresses which arise from restrained shrinkage.

Received by the Institute Dec. 30, 1957. Title No. 55-38 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 5, Nov. 1958 (*Proceedings* V. 55). Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Feb. 1, 1959. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

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A number of investigators have made a direct approach to the problem of restrained shrinkage and cracking, using methods similar to those adopted here, and their work has been reviewed elsewhere.[1] The results were not entirely conclusive and further research is justified. In particular there was little information on the stresses and strains which actually cause cracking under the slow rate of loading imposed by restrained shrinkage, and the present investigation was made to obtain further information of this sort.

MATERIALS AND TESTING PROCEDURE

Ordinary portland cement from a blend of five bags of one brand complying with Australian Standard A.2 was used. Fractions were taken from two natural quartz sands to give a gap graded material with a particle size distribution as shown below:

Sieve fraction (Tyler)	Percent by weight
8—14	61.4
14—28	—
28—48	—
48—100	25.8
100—	12.8

This rather unusual grading was chosen after a few preliminary tests because it was found to allow much easier compaction than either of the sands alone or a direct mixture of the two. The specific gravity of the sand used was 2.63.

Mixing and placing

Throughout the experiment the sand-cement ratio was kept constant at 3:1 by weight, but water-cement ratios by weight were 0.4, 0.5 or 0.6. The sand and cement were mixed dry in a pan mixer 30 sec, then the water was added and mixing continued for a further 2 min. After the material was placed in the molds it was compacted by vibration. The method of vibration and the time for which it was applied were varied according to the apparent requirements of each mix to achieve full compaction. Full compaction was assumed to have been reached when air bubbles ceased to rise from the mix and a continuous film of water just formed, but it was found subsequently that small amounts of air were retained in the 0.4 and 0.5 water-cement ratio mixes when this stage of compaction was reached.

Specimens

Three mixes were made for each water-cement ratio and from each mix 18 bars 1 x 1 x 11.25 in. were made. Twelve bars were used in pairs for the determination of the modulus of rupture at demolding and then at ages of 3, 7, 14, 21, and 28 days. Two bars were used for determination of the modulus of elasticity by flexural vibration at successive ages.

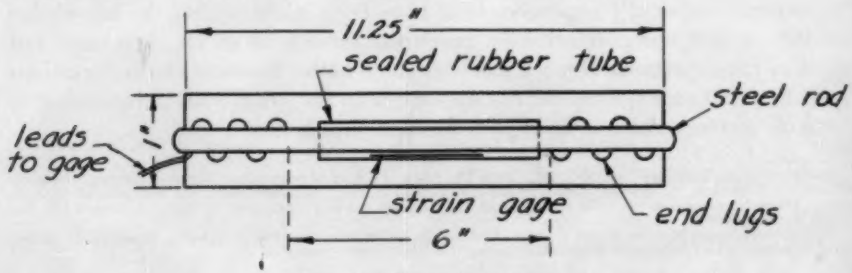

Fig. 1—The test specimen

Two bars with small stainless steel end plugs were used for the measurement of free shrinkage in air. Two bars with ¼-in. steel rods placed centrally through their length were used to examine the behavior of the mortar under partial restraint. Fig. 1 illustrates the type of specimen used.

Testing procedures

Specimens were demolded 22 hr after casting and stored in air at 70 F, 50 percent relative humidity. The output from the strain gage on the restrained bars was recorded on a chart every 3 min from the time of demolding. As shrinkage progressed the central rod was compressed and a continuously increasing strain was recorded. When the mortar cracked there was a sharp change in the strain reading, and it was found possible to read from the recorder chart the time to within 3 min at which the change in strain occurred.

Because there was no temperature compensating gage the strain gage record was not used for strain readings, but the actual contraction of the steel rod was measured in the dial gage comparator used for the free shrinkage measurements.

Readings of shrinkage, contraction, and modulus of elasticity were taken at suitable intervals of time. Specimens were weighed before measurement to determine the change of weight with time.

STRAIN CALCULATIONS

From the measured contraction of the steel rod the stresses in steel and mortar and the strain in mortar were calculated. It was assumed that the strain in steel and mortar increased linearly from a free end along the section in which the end lugs were situated, and was uniform over the center 6 in. in which the rod was encased in a rubber tube. This gave a trapezoidal strain distribution along the bar as shown in Fig. 2.

The measured contraction of the rod represents the average strain over the whole rod, so that for an assumed distribution the maximum strain over the gage length may be calculated. Alternative distributions of strain (e.g. parabolic) were assumed over the ends, but as these made little difference to the calculated maximum strain the linear distribution was adopted for simplicity.

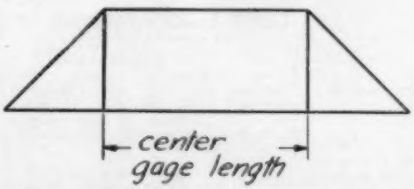

Fig. 2—Trapezoidal strain distribution along the bar

If ϵ_r = strain over gage length, then the corresponding stress

$$\sigma_r = E_s \epsilon_r$$

and since the tension T in the mortar must equal the compression C in the rod

$$A_s \sigma_r = A \sigma_t$$

where A_s is the cross-sectional area of the rod, A is the cross-sectional area of the mortar, and σ_t is tensile stress in mortar. Therefore

$$\sigma_t = \frac{A_s E_s \epsilon_r}{A} \qquad (1)$$

The elastic strain in the mortar may therefore be defined

$$\epsilon_e = \frac{\sigma_t}{E} = \frac{A_s E_s \epsilon_r}{AE} \quad \ldots\ldots (2)$$

where E is the value of the modulus of elasticity of the mortar determined by nondestructive testing at the appropriate time.

The strain ϵ_r represents the strain in the steel due to stress and it also represents the shrinkage of the restrained mortar. The strain ϵ due to stress in this mortar will be the difference between ϵ_r and the free shrinkage strain ϵ_s. That is,

$$\epsilon = \epsilon_r - \epsilon_s \quad \ldots\ldots (3)$$

This total strain ϵ may be considered to consist of an elastic and an inelastic or creep strain ϵ_c. Thus

$$\epsilon = \epsilon_e + \epsilon_c \quad \ldots\ldots (4)$$

ϵ is determined from Eq. (3) and ϵ_e from Eq. (2) so that ϵ_c can be calculated.

TEST RESULTS

The results from individual specimens are shown in Table 1. When the recorder failed for a period, the cracking times were noted as half way between the last reading at which the specimen was not cracked and the reading at

TABLE 1—CONDITIONS AT CRACKING OF 1:3 MORTARS

Mortar	Air content, percent*	Cracking time		Tensile stress, psi	Elastic strain, in. per in. $\times 10^{-6}$	Inelastic strain, in. per in. $\times 10^{-6}$	Unit shrinkage at time of cracking, in. per in. $\times 10^{-6}$
		Hr after drying commenced	Hr after shrinkage commenced (approximate)				
$W/C = 0.4$							
1st replicate	3.5	82.7	77.7	380	10.7	26.3	59.0
		28.0	23.0	300	8.2	8.4	34.0
2nd replicate	3.8	NC	NC	NC	NC	NC	NC
		124.3	119.3	440	10.3	30.7	66.5
3rd replicate	3.1	54.1	49.1	325	8.8	23.9	51.5
		153 ± 33†	148 ± 33	345 ± 5	9.4 ± 0.14	44.1	73.5
Average	3.5	88	83	358	9.5	—	56.9
$W/C = 0.5$							
1st replicate	2.0	120	110	240	9.3	44.2	67.5
		NC	NC	NC	NC	NC	NC
2nd replicate	3.3	48.2	38.2	315	12.3	17.0	47.5
		33.2	23.2	310	12.1	4.2	34.3
3rd replicate	1.7	61.7	51.7	296	10.3	25.7	53.2
		40.8	30.8	285	9.9	13.5	40.0
Average	2.3	61	51	289	10.8	—	48.5
$W/C = 0.6$							
1st replicate	0	87.5 ± 32†	72.5 ± 32	200†	11.4†	32.5‡	56.5‡
		52.3	37.3	225	12.8	19.6	45.5
2nd replicate	1.0	42.5 ± 5.5†	27.5 ± 5.5†	220 ± 15	11.4 ± 0.8	15.4	39.5
		39.7 ± 8.2†	24.7 ± 8.2†	280 ± 25	14.5 ± 1.3	7.4	37.5
3rd replicate	0	51 ± 3†	36 ± 3†	190 ± 5	8.9 ± 0.2	26.6	46.5
		40 ± 8†	25 ± 8†	190 $\begin{cases} +10 \\ -30 \end{cases}$	8.9 $\begin{cases} +0.4 \\ -1.3 \end{cases}$	15.4	35.3
Average	0.3	52	37	218	11.3	—	43.4

*Calculated from weight and volume of the specimen and the known properties of the constituents.
†Recorder did not function for a period.
‡Calculated from measurement prior to cracking.
NC = not cracked.

which it was first found cracked. Thus a cracking time recorded as 153 ± 33 hr means that the recorder failed for 66 hr and that the specimen was not cracked at 120 hr but had cracked at 186 hr. When this happened there was a similar uncertainty about the stress and the strain at failure and these results are also shown with appropriate tolerances.

There is a wide difference in the cracking times of specimens of the same mix and although the average cracking time decreases with increasing water-cement ratio these differences are not statistically significant. Later it will be shown that large differences in the time of cracking can be expected even for very small differences in the properties of the specimens. The decrease of tensile stress with increasing water-cement ratio is statistically significant at the 1 percent level. This was expected as an obvious consequence of increased strengths from lower water-cement ratios. In contrast to the tensile stress figures, the elastic strain is much more uniform. With inelastic strains there is again a very wide scatter of results.

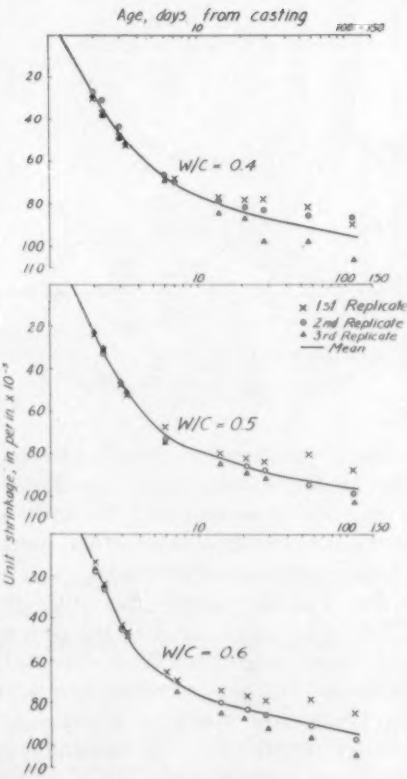

Fig. 3—Unit shrinkage with age

Fig. 3 shows the shrinkage of each mortar mix, each point representing the average results from two specimens. For a short period after demolding, varying with the water-cement ratio, the shrinkage is negligible; for the next few days the shrinkage is fairly rapid, but after 14 days the rate has decreased considerably and from then on is almost linear on a logarithmic scale.

Up to 7 days the differences of shrinkage between the three replications of each water-cement ratio are not great. The loss in weight curves (Fig. 4, 5, and 6) show the excellent agreement between the three replications of the 0.6 water-cement ratio, and between two of the three replications at the other two ratios.

Throughout this paper, change of weight of the mortars has been interpreted as change of water content. This is not strictly true because the weight is also affected by the reaction between the carbon dioxide of the air and cement, and the error will therefore depend on the extent of carbonation at any time. There are insufficient published data on the rate of carbonation of mortars to

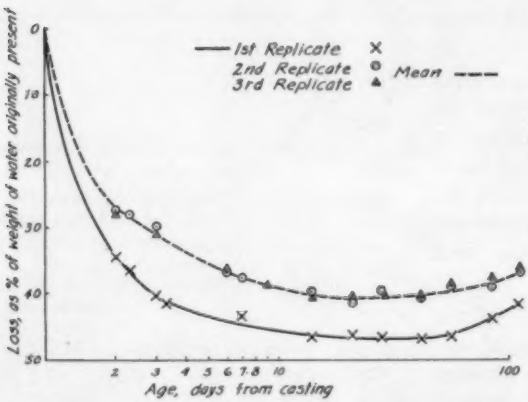

Fig. 4—Loss in weight with age, W/C ratio 0.4

allow a quantitative correction to the weight losses for this effect. Although the quoted water contents in the mortar may be in error absolutely, it is reasonable to assume that the error is small at early ages and that the values given do provide a satisfactory basis of comparison between the mortars of different water-cement ratios.

Fig. 7 shows the variation of the dynamic modulus of elasticity with time. There is a greater variation between successive replications than for shrinkage and loss of weight, and it appears that this modulus reaches a maximum at an early age and then decreases slowly up to 50 days, after which it rises again. No great error would be introduced into calculations if it were assumed to remain constant once the maximum has been reached. The point at which the maximum is reached is not well defined, but for the water-cement ratio of 0.4

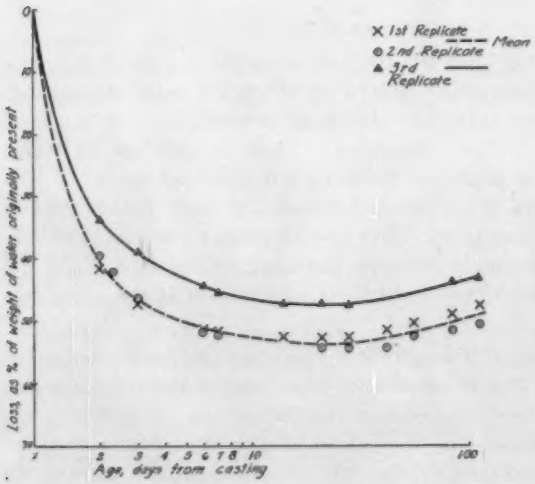

Fig. 5—Loss in weight with age, W/C ratio 0.5

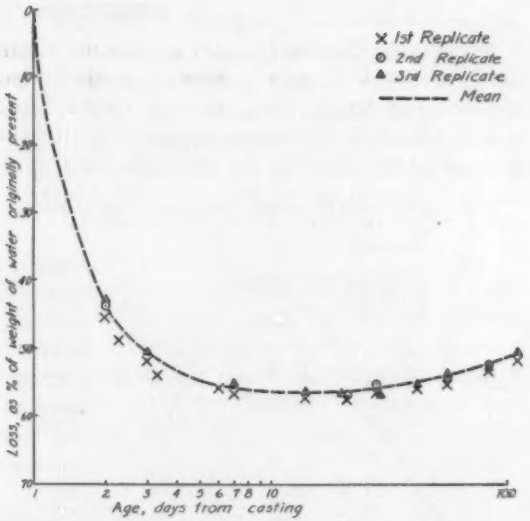

Fig. 6—Loss in weight with age, W/C ratio 0.6

it is probably between 1 and 2 days after casting; for $W/C = 0.5$ it is close to 2 days, and for 0.6 it is between 2 and 3 days or possibly later.

The progress of the modulus of rupture with time up to 28 days is shown in Fig. 8.

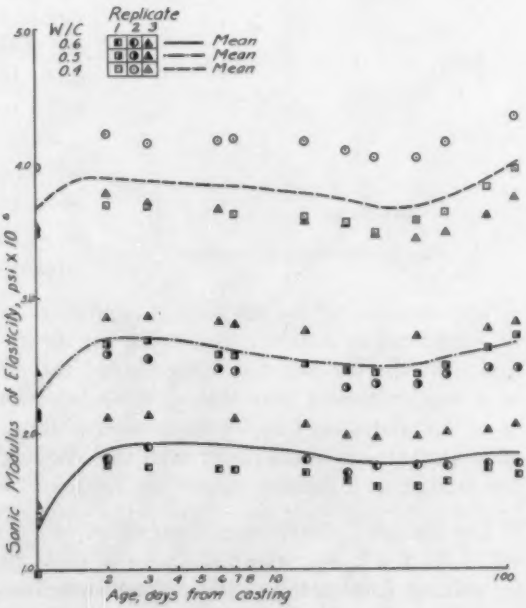

Fig. 7—Modulus of elasticity

DISCUSSION

The results given in Table 1 support an hypothesis that the mortar cracks at an average elastic tensile strain of 11×10^{-5} independent of inelastic strain, water-cement ratio, and age up to 7 days. As the mortar is a heterogeneous material, this strain would be the strain at which either the hardened cement paste breaks or the adhesion between cement paste and sand fails.

Before this study some preliminary experiments on the restrained shrinkage of set cement paste were made. These experiments were done without the automatic recorder to measure the time of cracking and showed a very wide scatter of results, but the average elastic strain in the cement paste at failure was about $25 \times 10.^{-5}$ The difference between this strain and the cracking strain of the mortar may be explained in two ways. Either, as suggested above, the mortar strain is that at which the cement paste matrix and the sand separate, or alternatively, the shrinkage of the cement paste around the sand causes an elastic strain of about 14×10^{-5} in the paste so that a further applied strain of 11×10^{-5} would cause cracking of the paste. It is not difficult to show that this latter explanation is quite feasible, although no estimate of its probability can be made without much more extensive data on the creep properties of set cement paste at an early age, and on the elastic properties of the aggregate.

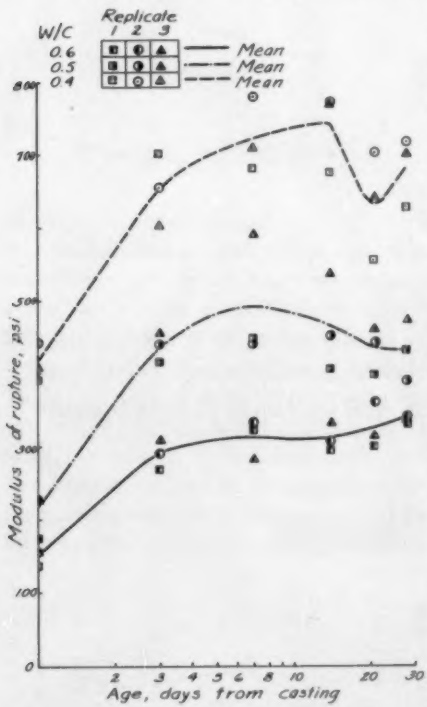

Fig. 8—Modulus of rupture

The variation of the modulus of rupture with time as shown in Fig. 8 may be considered as evidence supporting the theory that the cement paste cracks first. At the 0.4 and 0.5 water-cement ratios the modulus of rupture rises to a maximum and then drops. This behavior is most marked at 0.4, less so at 0.5, and absent at 0.6 water-cement ratio. Such a decrease in strength after a maximum could result from the cracking of the cement paste phase of the mortar, as it shrinks against the restraint offered by the aggregate.

The restraint offered by the aggregate to the paste shrinkage will be greatest in the 0.4 mixes, where the ratio of paste to sand is approximately 1:1.57 by volume, and least in the 0.6 mixes, where the ratio of paste to sand is about

1:1.23, so that the probability of cracking of the cement paste in mortars increases as the water-cement ratio decreases. It is impossible to provide a quantitative expression of this behavior because of the difficulty—perhaps impossibility—of measuring the modulus of elasticity of the sand grains, and of producing cement paste specimens with water-cement ratios of the order of 0.5 and 0.6. Bleeding would undoubtedly alter the effective proportions of pastes mixed with high water-cement ratios so that the properties thus determined would have little relation to the paste phase of a mortar which it was intended to simulate.

From a practical point of view it is necessary to be able to calculate whether or not the elastic strain of 11×10^{-5} will be reached under given conditions of restraint. This requires knowledge of the creep characteristics of the material. Attempts were made to find an expression for the inelastic strain in terms of stress and time but generally the data were insufficient. It appeared that the form of the expression varied greatly for different mixes and therefore had little practical value.

Relationship between creep and shrinkage

Several workers (e.g., Lynam[2]) have suggested that there is a relation between creep and shrinkage. Fig. 9, 10, and 11 show plots of shrinkage against creep, or inelastic, strain. For the first 2 to 2½ days the rate of creep with shrinkage is low, but after this time it increases. In the latter period the rate of creep appears to be independent of water-cement ratio; the gradient of the curves at this stage is about 0.8. The average elastic cracking strain for

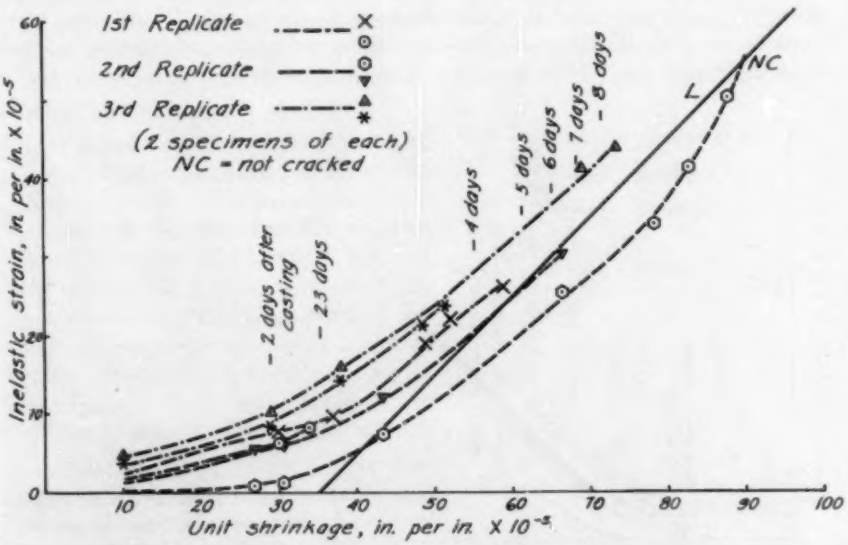

Fig. 9—Relation of inelastic strain to unit shrinkage, W/C ratio 0.4

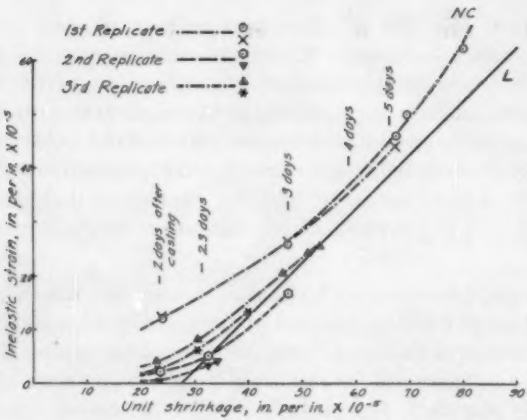

Fig. 10—Relation of inelastic strain to unit shrinkage, W/C ratio 0.5. Two specimens of each. NC = not cracked

the mortars is 11 × 10.$^{-5}$ For each water-cement ratio it is possible to calculate the shrinkage which would produce this elastic strain in the absence of creep. In each of Fig. 9, 10, and 11 a line is drawn with gradient of unity and passing through this calculated value of shrinkage. Any curve parallel to this line L represents a condition in which all the continuing shrinkage is being dissipated by creep. The line L itself represents the balance between creep and shrinkage for an elastic strain equal to the cracking strain 11 × 10.$^{-5}$ Any specimen should crack when its curve intersects the line L. It can be seen that only a small change in the rate of creep over the first 2 days would alter markedly the point at which a particular curve cuts the line L, and therefore changes greatly the time at which cracking might occur. Thus, even when there is very little difference between most properties of specimens their cracking times may differ greatly. This feature is shown more simply in

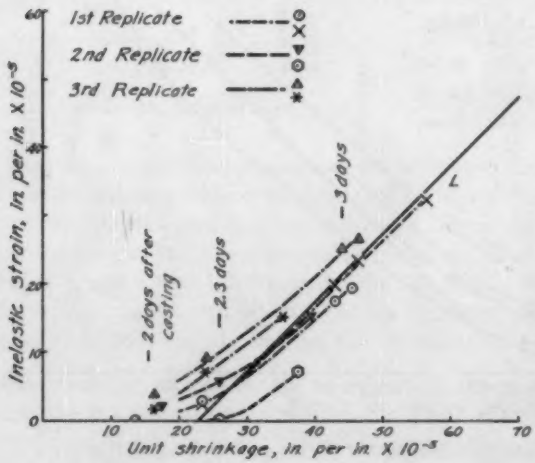

Fig. 11—Relation of inelastic strain to unit shrinkage, W/C ratio 0.6. Two specimens of each

Fig. 12 which illustrates the growth with time of elastic tensile strain for the restrained specimens. It can be seen that most of the curves rise steeply for a short time, after which the gradient rapidly decreases until they are almost parallel to the line *U* of average ultimate strain, and that small differences in gradient in this latter stage would greatly alter the time at which ultimate strain is reached.

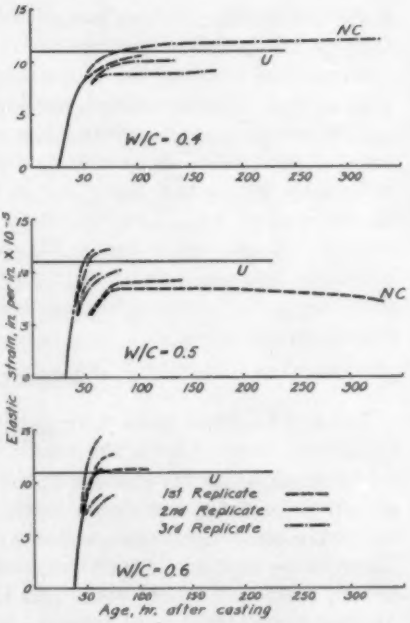

Fig. 12—Elastic strain. Two specimens of each. *U* = average ultimate strain. NC = not cracked

Influence of moisture content

Instead of considering the loss of water of the mortars, as is done in Fig. 4, 5, and 6, the emphasis may be placed on the water retained by the specimens. This may be expressed as a percentage by volume of the cement paste phase of the mortar, and the variation with time of this water held is shown in Fig. 13. The results are the average of the three replicates for each water-cement ratio. About 12 hr after demolding there was no significant difference between the amounts of water remaining in the specimens of each of the three water-cement ratios.

It would be expected that the water loosely held in capillaries would be the first to be lost. This has no influence on the volume change of the material until the water held has been reduced to between 42 and 46 percent of the paste volume. The time at which the rate of creep with shrinkage increases cannot be exactly defined but it would correspond to a water content of about 36 percent of the paste volume.

Other writers (Neville[3]) have suggested that creep may be associated with a redistribution or change of moisture content of concrete. As a

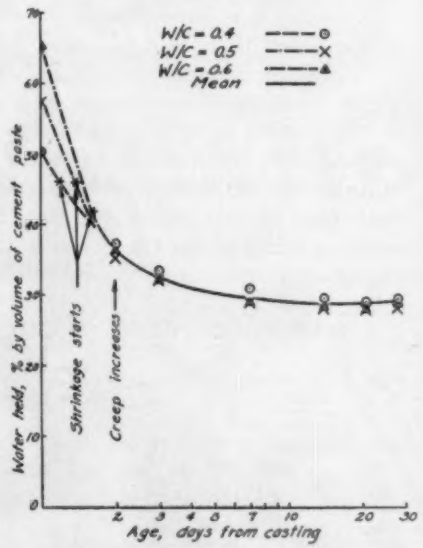

Fig. 13—Water held in cement paste

slight elaboration of this theory it is suggested that the rate of creep for the early shrinkage is low because the deformation of the gel is restricted by the movement of water in the capillaries, which contain more water at this stage than later. The volume of capillaries will increase with increasing water-cement ratio[4] so that a greater loss of water should be expected for a higher water-cement ratio. At a certain level of water content the capillaries become sufficiently empty that the water in them no longer impedes the deformation of the cement gel. This condition is represented by the steeper gradient section of the curves in Fig. 9, 10, and 11. As there is little or no difference in the constitution of the cement paste in the three cases, the relation between shrinkage and inelastic deformation becomes independent of the initial water-cement ratio.

ADDITIONAL TESTS

A few additional tests were carried out to examine further some of the hypotheses derived from the results of the main series of tests. So far, the evidence seems to favor slightly the theory that cracking in the restrained specimens begins in the cement paste phase. If this is correct then the cracking strain should decrease as the amount of sand in the mortar increases. Three mixes were made therefore; one of pure cement paste, one with a sand-cement ratio of 1.5:1 by weight, and a third with a sand-cement ratio of 3:1. All had the water-cement ratio 0.4 by weight, and two restrained specimens similar to those used previously were made from each mix and treated in the same way as in the main tests. Results of these supplementary tests are set out in Table 2.

Although it is impossible to draw definite conclusions from so few tests it appears that there is a tendency for the elastic cracking strain to decrease with increasing aggregate content. This supports the theory that the cracking starts in the cement paste, as it shows that greater strains are developed in the cement paste with increasing restraint of the paste by the aggregate and the extra strain required to cause fracture decreases.

Note that the elastic cracking strain of the pure cement paste is much less than that quoted earlier (p. 594) from preliminary tests, and the elastic cracking strain of the 1:3 mortar is also much less than that for a similar mix in the main experiment.

TABLE 2—CONDITIONS OF CRACKING AT 0.4 WATER-CEMENT RATIO

Mix	Air content, percent	Time of cracking, hr	Stress, psi	Elastic strain, in. per in. $\times 10^{-6}$	Inelastic strain, in. per in. $\times 10^{-6}$	Unit shrinkage at cracking, in. per in. $\times 10^{-6}$	Shrinkage starts after:	Water-cement ratio by weight when shrinkage starts
1:0	0	31—48* 21—31*	180 100	14.2 8.2	48.2 11.0	25—73* 5—25*	12 hr	0.32
1:1½	0	37.3 41	172 172	7.1 7.1	17.9 21.9	30 36.5	5 hr	0.34
1:3	3.25	31—98* 23	180 180	5.4 5.4	0 5.7	25—53.5* 21.5	5 hr	0.33

*Cracked within the range given.

The cracking times in these additional tests are generally much less than those noted earlier for 0.4 water-cement ratio, and these low results may indicate that the cracking strain is time dependent, at least at very early ages.

The cement for these supplementary tests was of the same brand as that used in the main experiment but from a different batch, and this may also contribute to the variation in results.

The water-cement ratio at the start of shrinkage ranged from 0.32 to 0.34, which represents a water content held in the paste of between 44 and 47 percent. This checks fairly well with the range from 42 to 46 percent (Fig. 13) for the main experiment. That this water content is roughly constant suggests that there may be two distinct water phases, as in timber, only one of which influences shrinkage. It is, for example, conceivable that the difference is between capillary water and gel water, but again this remains to be proved.

CONCLUSIONS

1. The most likely criterion of failure for mortars under uniaxial stress arising from restrained shrinkage is maximum elastic tensile strain. Within the range covered by the main experiment the criterion is independent of water-cement ratio and time of failure. There is some indication that the cracking strain may be time dependent at very early ages.

2. Although the evidence is not conclusive it appears most likely that the cracking during air drying starts in the cement paste and not in the bond between paste and aggregate.

3. If the water retained in the 1:3 mortar at any age is expressed as a percentage by volume of the cement paste phase, this percentage becomes independent of the initial water-cement ratio within a few hours of the commencement of drying.

4. No appreciable shrinkage occurs in the mortar until the water retained has been reduced to about 45 percent of the volume of the cement paste phase.

5. The creep in the mortars is low for the first two or three days of the drying period but then increases.

ACKNOWLEDGMENT

The author acknowledges gratefully the considerable assistance of R. K. Lewis in the conduct of all of these tests and the analysis of the results.

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Title No. 55-39

BLAST AND EARTHQUAKE RESISTANT DESIGN DATA:

behavior of one-story reinforced concrete shear walls containing openings*

By JACK R. BENJAMIN and HARRY A. WILLIAMS†

A number of scale model one-story reinforced concrete shear walls containing openings were tested as a part of a major study of shear walls. This paper reports on the observed behavior, theoretical studies, and recommendations for analysis of such walls. Studies of the influence of variations in reinforcing are included.

■ A general investigation of the strength and behavior of shear walls, some of which contained openings, was conducted at Stanford University during 1951-56. The work reported herein was performed under Contract DA 49-129-Eng-193 with the Office of the Chief of Engineers, Department of the Army. Many of the theoretical studies reported herein were made by Robert H. Stivers.[1] Complete discussions of all work undertaken are to be found in two reports by the authors.[2]

The purpose of the investigation was to develop data on the behavior of shear walls containing openings which could be used in the design of structures to resist atomic blast loads. The behavior of concrete walls with solid panels is discussed in another paper by the authors.[3] The approximate analysis of shear walls containing openings is discussed in a bulletin by the Portland Cement Association.[4] Several Japanese investigators have also studied the problem.[5-10]

This paper is concerned with the behavior of one-story reinforced concrete shear walls containing openings. The general types and the loading arrange-

*Based on a paper presented at the tenth ACI regional meeting, Seattle, Wash., Nov. 5, 1957. Title No. 55-39 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 5, Nov. 1958, *Proceedings* V. 55. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Feb. 1, 1959. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

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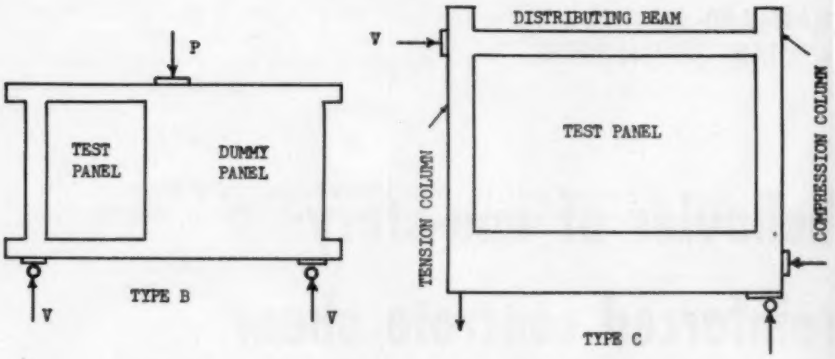


Fig. 1—Shear wall specimens and methods of loading

ments are shown in Fig. 1. The wall is loaded through a distributing member at the top of the wall. This may be a beam or a floor diaphragm. Both tension and compression columns are provided and the wall is supported on an essentially rigid foundation. The tension and compression columns may have the same thickness as the panel but are provided with special steel in any case. Special steel is provided around the wall openings.

Two different investigations were made. The first study was concerned with means of analyzing and predicting the behavior of shear walls containing openings of various types. Special steel was provided around the openings. It was kept constant in quantity and was similar to conventional practice. The objective of the second series of tests was to study the influence of opening steel of various types. In the first study, openings were found to decrease both strength and rigidity compared to a solid panel. The second series of tests was then designed to see if these influences could be modified by changes in the special steel around the openings.

TABLE 1—OPENINGS IN PANELS OF H-SERIES MODELS
(See Fig. 3)

Specimen No.	Number and type of openings	Height and width of opening, in.	Location, in.		Remarks
			a	*b*	
H-1 and H-3	One window	11 x 19	19.5	11.5	See Fig. 6 and Note 1
H-2 and H-4	One door	22 x 19	19.5	11.5	See Fig. 6 and Note 1
H-5 and H-5a	One window	6 x 6	26.0	13.75	See Fig. 6
H-6	One window	6 x 6	40.5	22.1	See Fig. 6
H-7	First door	22.4 x 13.6	8.6	11.1	See Fig. 6
	Second door	22.4 x 13.6	35.8	11.1	
H-8	One window	11.2 x 13.6	8.6	11.1	See Fig. 6
	One door	22.4 x 13.6	35.8	11.1	
H-9	First door	26 x 22.2	0	7.5	
	Second door	26 x 22.2	30.5	7.5	

Notes:
1. All panels except H-1 and H-2 were reinforced with #2 bars at 4.9-in. centers both ways (0.5 percent reinforcing).
2. Two additional #2 bars were provided around all openings.

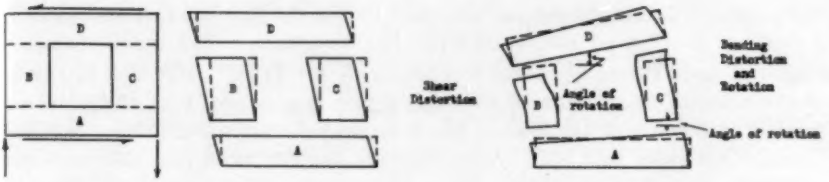


Fig. 2—Schematic representation of distortions of shear wall opening

Considerable variation in experimental results always occurs. The problem then is to find means of predicting the mode of behavior and the load-deflection curve within reasonable limits.

MATHEMATICAL INVESTIGATIONS

Extensive mathematical studies were made during the program.[1] They ranged from the approximate Portland Cement Association procedure[4] to approximate theory of elasticity solutions of the walls using the lattice analogy.[11,12] The lattice analogy solutions gave the best results and the best insight into the wall behavior. However, the procedures are most complex and time consuming. The Portland Cement Association procedure is satisfactory in predicting the shear distribution between piers but otherwise unsatisfactory. This solution considers only the bending and shearing distortion of the individual piers. The wall rigidities computed are much too large and strength calculations are in error largely because axial stresses in the piers are neglected.

A satisfactory engineering solution for the linear range of behavior is possible using the methods of strength of materials. A complete statical solution is necessary and all sources of deflection must be considered. The

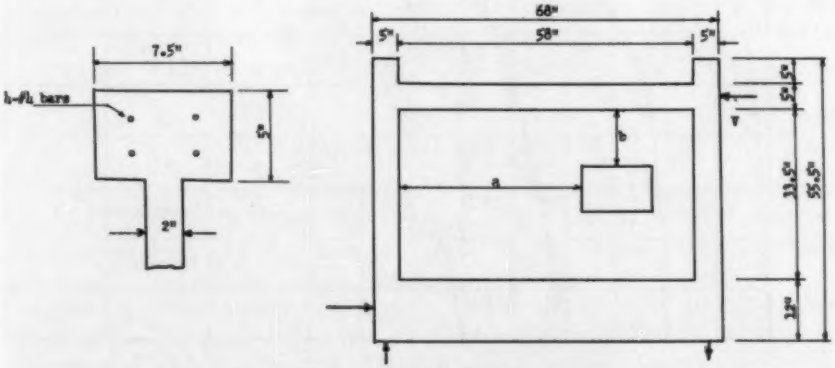


Fig. 3—Details of H-series specimens. Left—Typical section, top beam and columns. Right—Dimensions

distortions to be considered are sketched in Fig. 2. The bending and shearing distortion of all wall elements must be calculated. The axial change in length of piers B and C causes a rotation of pier D and deflection of the top of the wall. The rotation of the top of pier A produces further lateral distortion of the top of the wall. All of these distortions are small quantities. Calculations show that with many walls the bending, shearing, and rotational influences are all of the same relative magnitude. The load at the break in the deflection curve or the limit of elastic action can also be calculated satisfactorily using methods of strength of materials and a tensile strength in the concrete of 0.10 f_c'.

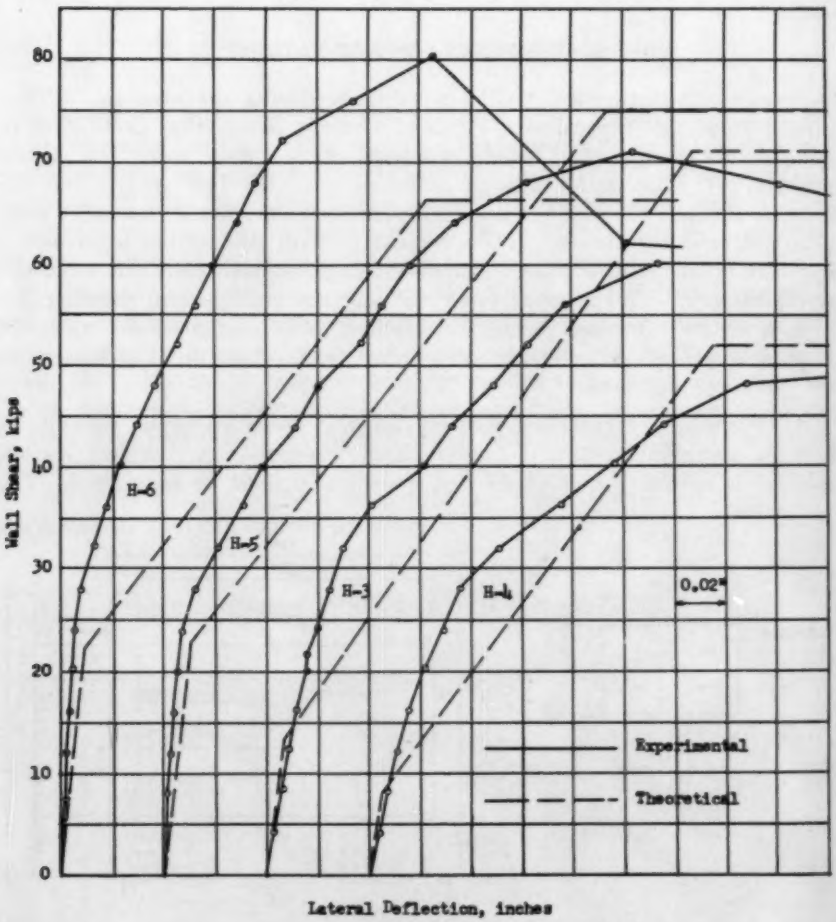


Fig. 4—Load-deflection curves for H-series walls H-3 to H-6

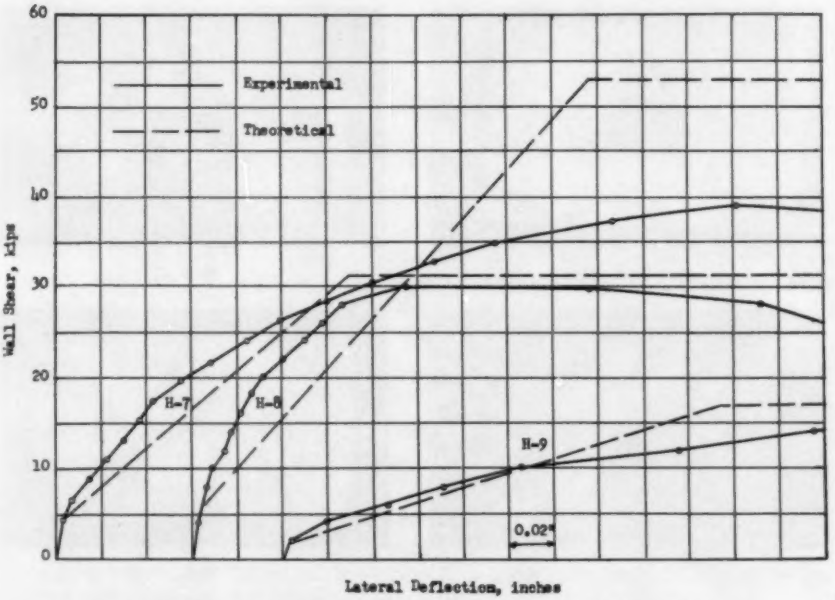


Fig. 5—Load-deflection curves for H-series walls H-7 to H-9

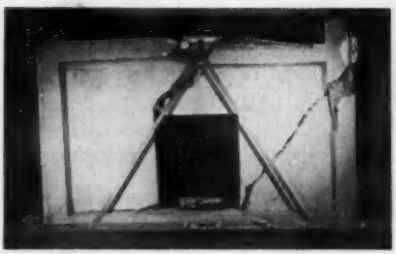
Ultimate load calculations are subject to considerable error. Best results are obtained assuming the total wall strength is the sum of the strengths of its individual piers. The strengths of the piers in shear can be calculated[3] as well as that in bending. Bending strength calculations are made assuming the tension column steel yields with the section cracked through to the compression steel. The smaller of the two strengths is then the pier strength.

The deflection at ultimate load was found experimentally to be approximately five times the elastic deflection that would be present if the wall acted elastically up to the ultimate load. With all walls the maximum crack width was of the same order of magnitude as the wall deflection at the same load.

EXPERIMENTAL PROCEDURES

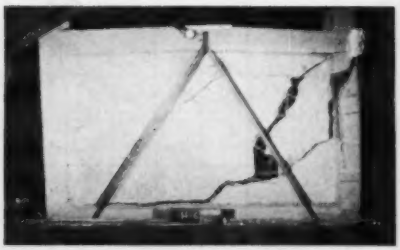
A total of 21 model walls were tested. Three model walls had solid panels for correlation with those containing openings. All walls were of approximately quarter scale if a prototype with an 8 in. thick wall is considered. The particular studies in this report do not depend on scaling. Each wall can be considered a structure in itself. While extrapolation to full size walls may introduce some scale effect, experience indicates that it should be no larger than the normal scatter of results to be expected. An error in prediction of 10-20 percent is to be expected with shear walls with individual cases being as much as 50 percent from the mean.

H-3



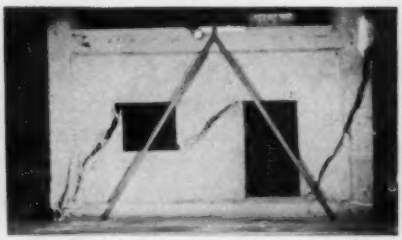
H-4

H-5a



H-6

H-7



H-8

Fig. 6—Selected H-series specimens after testing

The two types of walls tested are shown in Fig. 1. Type B walls were tested in a 200,000-lb Riehle testing machine. Type C walls were tested in a specially constructed shear jig.[3]

Type I portland cement was used in the concrete mix with a small amount of additive to improve workability. Walls were damp cured for 1 week and then allowed to dry. The walls were tested after curing approximately 2 weeks when the concrete strength was of the order of magnitude of 3000 psi. Actual strengths varied from 2510 to 3840 psi. The reinforcing steel was of structural or intermediate grade. All bars #3 and over were deformed while those #2 and smaller were smooth except for one series of tests where the #2 bars were slightly deformed. These deformations had no apparent influence on the test results.

The test procedure consisted of applying a predetermined load and then reading all gages. The load level remained essentially constant during the reading of the gages as long as the concrete was uncracked. Once major cracking began the load tended to drop off somewhat during the recording period.

TESTS OF TYPE I WALLS

Two series of wall tests were run to determine the behavior of walls containing openings of conventional construction. Walls H-1 to H-9 were identical except for the size and shape of openings. Walls H-1 and H-2 had unreinforced panels and will not be discussed here. Details of construction are given in Fig. 3 and Table 1. Load-deflection curves are given in Fig. 4 and 5, and typical photographs of specimens under test in Fig. 6.

Walls H-3 and H-4 contained window and door type openings. With both specimens the pier on the tension end of the wall failed in shear while that on the compression side failed in bending. Predicted curves show good agreement with experimental results. Walls H-5 and H-6 were designed to investigate the influence of small openings and the location of that opening. Predicted curves are based on a wall with a solid panel neglecting openings. Specimen H-5 shows excellent agreement between experiment and theory while that for H-6 is not as good. It is likely that the location of the opening in H-6 actually increased the ultimate strength over that of a solid panel because of the crack pattern in the wall produced by the opening.

Specimens H-7, H-8, and H-9 all contained two openings. H-7 approximates two doors symmetrically placed. One of the doors was changed to a window in H-8 and the limit of two large doors was approached in H-9. Elastic deflections show excellent agreement between calculated and experimental results. The predicted curves are excellent for H-7 and H-9 while the predicted ultimate load for H-8 is very much in error. Such errors should be expected because of the complex failure patterns involved. Specimen H-7 exhibited both bending and shear failures; H-8 failed in shear; and H-9 (not shown) appears to have

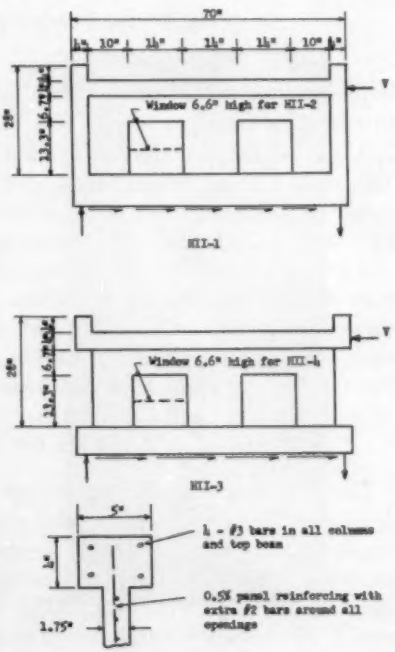


Fig. 7—Details of HII-series specimens

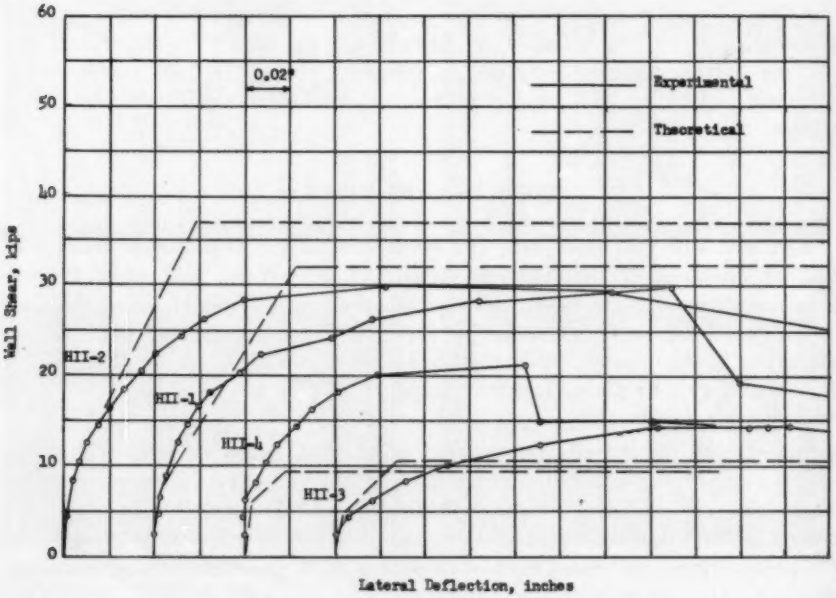


Fig. 8—Load-deflection curves for HII-series walls

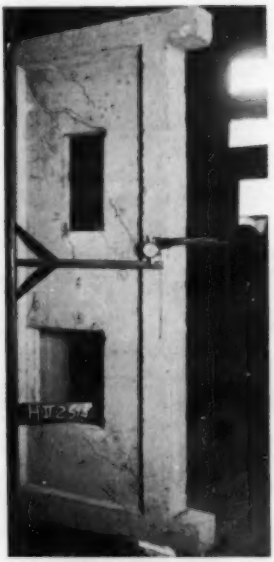
both bending and shear failure although calculations indicate bending to be the most critical.

A second series of four walls was tested with different wall proportions. The second series, Walls HII-1 and HII-2, had a much larger length-height ratio with openings in the same ratio to the total wall proportions as walls H-7 and H-8, respectively. Walls HII-3 and HII-4 were identical to HII-1 and HII-2, respectively, except that the main tension and compression columns were eliminated. Details of construction are shown in Fig. 7. Load-deflection curves are given in Fig. 8 and test specimens in Fig. 9. Agreement between theoretical and experimental results is excellent in the uncracked range but less satisfactory insofar as the prediction of ultimate load is concerned.

TESTS OF TYPE II WALLS

The first program of tests showed that openings reduce both strength and rigidity. These walls were reinforced around the openings according to a simple rectangular pattern with two #2 bars anchored according to conventional practice. Such reinforcing corresponds to two #8 bars used as opening steel in an 8-in. wall. (This compares with two #5 bars as ordinarily used.) The panels were reinforced uniformly with a steel ratio of 0.005 in both directions. This ratio was selected to emphasize the influence of the panel steel.

HII-1



HII-2

HII-3

HII-4

Fig. 9—HII-series specimens during testing

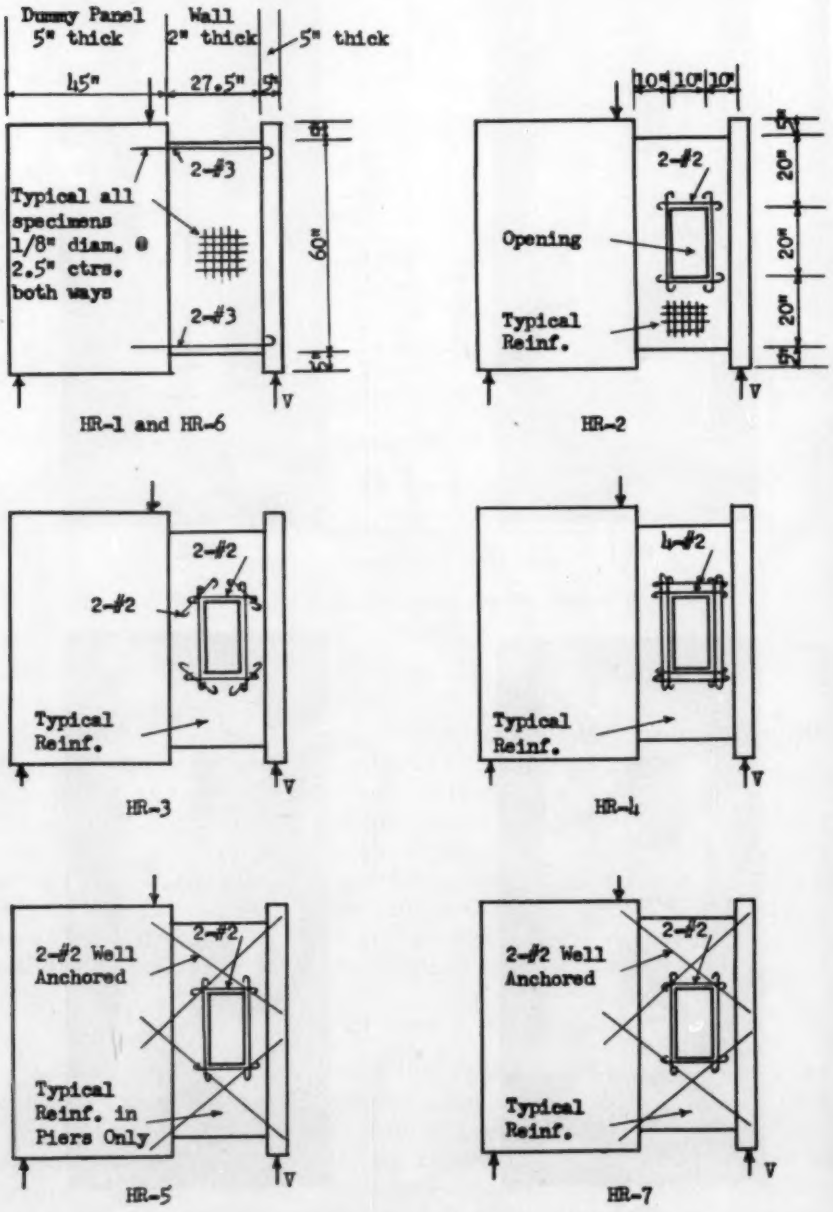


Fig. 10—Details of HR-series specimens

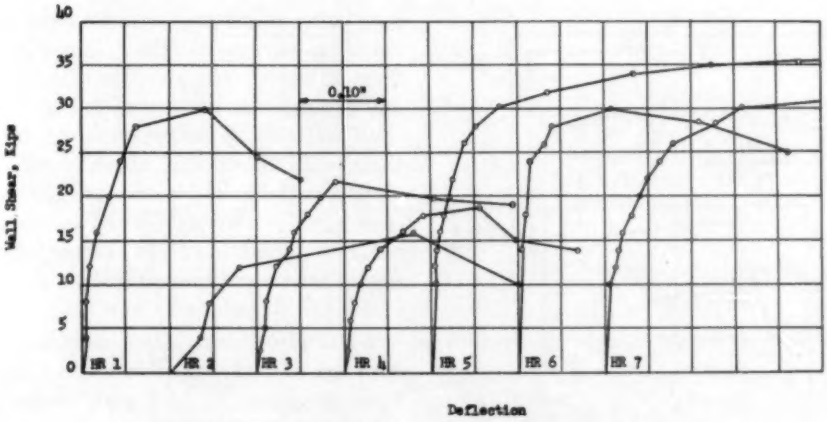
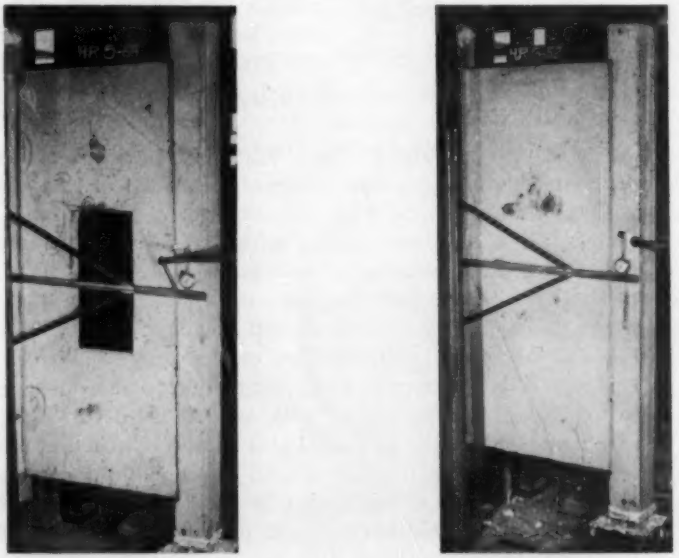


Fig. 11—Load-deflection curves for HR-series walls

Ten walls were constructed to investigate the influence of various types of special opening steel on rigidity and strength. Walls HR-1 and 6 contained a solid panel while HR-2, 3, 4, 5, and 7 had various opening steel patterns. Details of construction are shown in Fig. 10. The opening covered the central 1/9 of the panel area. Wall HR-2 had two #2 hooked opening bars. Two short diagonal #2 hooked corner bars were added with HR-3. The opening



HR-5 HR-6

Fig. 12—HR-series specimens during testing

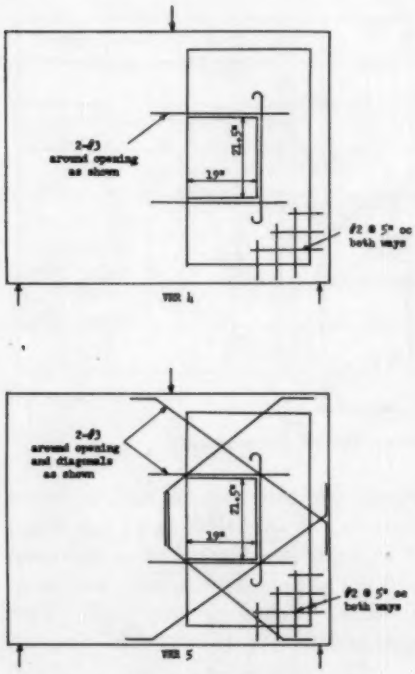


Fig. 13—Details of specimens VRR-4 and VRR-5

steel with HR-4 was four #2 bars in the same pattern as HR-2. Load-deflection curves are given in Fig. 11, and typical tests illustrated in Fig. 12. Study of the load-deflection curves for these walls shows that all three reinforcing patterns had a smaller rigidity than a wall with a solid panel and also reduced strength. The corner diagonal bars were more effective than additional straight side bars.

Walls HR-5 and HR-7 were then designed to study this trend. In HR-5 the usual two #2 bars were used around the opening but two #2 long diagonal bars were added in each pier, crossing the corners of the opening and securely anchored into both the foundation and loading beam. The pier steel ratio was also increased to 0.010. Wall HR-7 was identical to HR-5 except that the pier steel ratio remained at 0.005 the same as the remainder of the panel. These two walls behaved in a similar fashion. Compared to HR-1 and HR-6, walls with a solid panel, these two walls showed no appreciable reduction in rigidity or strength due to the opening.

A second series of tests was run to check this result on a wall of somewhat different proportions containing a door covering 21 percent of the panel area. Construction details are given in Fig. 13. Specimens VRR-1 had a solid panel; VRR-4 contained the door opening with two #3 rectangular opening bars; and VRR-5 had these rectangular bars as well as two #3 long crossed diagonal bars in each pier. Load-deflection curves are given in Fig. 14 and typical photographs of VRR-5 under test will be found in Fig. 15. Walls VRR-1 and 5 had identical load-deflection curves while that for VRR-4 showed both a reduction in strength and rigidity due to the opening. It is interesting to note that the first crack in the solid panel occurred at a wall shear of 32 kips while first cracks in VRR-4 and VRR-5 occurred at a shear of 16 kips. The first crack in VRR-4 produced a pronounced disintegration pattern while the long diagonal reinforcing in VRR-5 appears to have restricted this disintegration, producing a final load-deflection curve close to that of VRR-1.

These two series of tests definitely show the importance of long well-anchored rectangular and diagonal reinforcing around an opening. Further

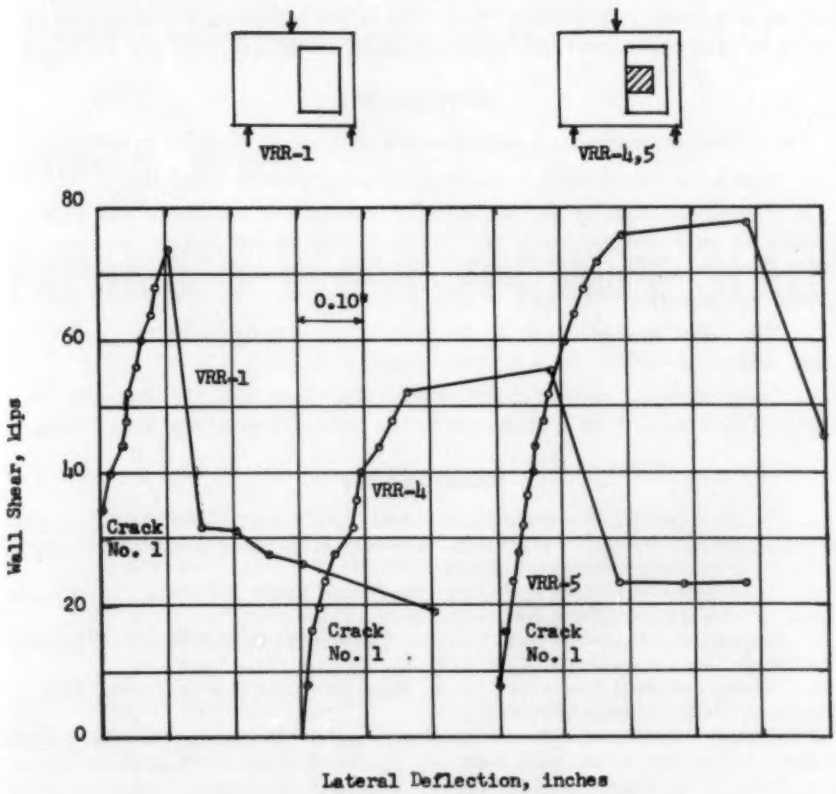


Fig. 14—Load-deflection curves for VRR-series walls

Fig. 15—Specimen VRR-5 during and after test

testing is necessary to confirm the extent of the influence of this special steel in walls with other sizes and locations of openings.

CONCLUSIONS

The following conclusions were reached as a result of these investigations:

1. The behavior of walls containing openings can be predicted.

2. The application of the methods of strength of materials will produce results of sufficient accuracy for design in the elastic region of behavior. All deformations from bending, shearing, and axial loading must be included in the computation.

3. The total wall strength is the sum of the strengths of the individual piers assuming each to be a separate shear wall.

4. Long diagonal well-anchored corner bars significantly increase both strength and rigidity of a wall containing openings once cracking begins.

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Title No. 55-40

lightweight concrete made with EXPANDED BLAST FURNACE SLAG*

By D. W. LEWIS†

SYNOPSIS

The tests reported provide data on both insulating and structural concretes made with typical expanded slag aggregates. The test data provide information on strength, durability, heat transmission, and unit weight of concretes made with various cement contents and amounts of entrained air. The effects of different aggregate top sizes and of various natural sand substitutions for the fine aggregate are discussed.

LIGHTWEIGHT CONCRETE has been successfully employed in both precast and cast-in-place concrete for many years. In recent years more designers are taking advantage of the reduced weight which permits savings in materials for foundations and load carrying members, and the superior characteristics in fire resistance, thermal insulation, and acoustical properties.

Many types of lightweight aggregate have been used, including expanded blast furnace slag, in the production of lightweight concretes for masonry units and insulating and structural concretes. Although use in concrete block[1] provides the major outlet for expanded slag, its utilization in structural lightweight concrete is increasing. In recent years a number of large buildings have used expanded slag concrete to obtain significant reduction in the dead load.

Expanded slag is a lightweight, cellular mineral aggregate produced by treating molten iron blast furnace slag with controlled quantities of water, often combined with compressed air and/or steam. It was first introduced in the United States about 1928. There are several methods for bringing the slag in contact with the proper quantities of water. These procedures, including the pit or jet process, the Kinney-Osborne, and the Caldwell and

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Brosius machines have been described by Miller.[2] All processes result in the same end product—a lightweight clinker, subsequently processed by crushing and screening to required sizes.

Earlier investigations of lightweight concretes made with expanded slags included the work of the Bureau of Reclamation[3] and the National Bureau of Standards.[4] Results of these investigations were combined in a single publication on lightweight aggregate concretes by the Housing and Home Finance Agency.[5]

Typical present-day expanded slags vary considerably in gradation, unit weight, and physical properties from some of those tested in this early work, indicating a need for more up-to-date data. Some question also exists as to whether all materials included in the earlier tests were actually expanded slags. In fact, one reference (Reference 3, p. 594) lists two of them as "granulated" rather than expanded.

Available data on lightweight aggregates and structural lightweight concretes have been summarized by Washa[6] and Kluge.[7] Data on creep, coefficient of expansion, strength, and other properties of 3000- and 4500-psi expanded slag concretes are contained in a recent paper by Shideler.[8] Results of these, as well as earlier tests, indicate that expanded slag concretes have somewhat higher moduli of elasticity than do those made with other lightweight aggregates. This characteristic is important in decreasing deflections of lightweight load bearing members.

Other references containing data on the heat transmission and fire resistance properties of lightweight concrete made with expanded slag aggregates include Foxhall's discussion of heat transmission[9] and the National Board of Fire Underwriters reports on fire tests[10] and fire resistance ratings.[11]

PURPOSE AND SCOPE OF INVESTIGATION

Although the use of expanded slag aggregates in lightweight concrete has been increasing, the data available on characteristics of concrete mixes made with typical materials and covering a wide range of cement contents and unit weights for the same aggregate are quite limited. Similarly, data on the effects of aggregate size and the use of natural sand in the mixes are lacking.

The research program reported in this paper was therefore undertaken to obtain data on the strength, durability, unit weight, thermal conductivity, and other characteristics of concrete made with typical expanded slag over a wide range of cement contents (4 to 8½ sacks per cu yd to include both insulating and structural concretes). A wide range of air contents was used, and the work included a limited investigation of the effect of different top sizes of aggregates. Similar data, limited to structural concretes and using a single top size of slag was obtained on mixes with varying amounts of natural sand as part of the fine aggregate. Coefficient of thermal expansion and drying shrinkage tests were incorporated in this series.

Tests on two expanded slags constituted the bulk of the work, although concretes made with two others were subjected to thermal conductivity tests.

No evaluation of the effects of different cements or of admixtures was considered in the test series.

MATERIALS AND TEST PROCEDURES

Expanded slag aggregates

The two expanded slags employed for the major portion of the testing program, designated as A and B, were selected as the result of extensive tests on the physical properties of expanded slags from all available sources. They are believed to be typical of such materials now being produced, irrespective of the expansion method used.

Slag A was used for the main test series of both insulating and structural concretes with all slag aggregate, and for the investigation of effect of top size of aggregate. In addition to strength tests, freezing and thawing and thermal conductivity tests were made on the concretes containing Slag A. Results of physical tests are shown in Table 1. Included are the gradations and unit weights of the combined coarse and fine aggregates as used in the concrete mixes. Dry loose unit weights of the ⅜ in. to No. 4 and No. 4 to 0 sizes were 41.7 and 63.0 lb per cu ft, respectively.

Slag B, with dry loose unit weights of 35.8 and 51.2 lb per cu ft in the ⅜ in. to No. 4 and No. 4 to 0 sizes, respectively, was used in structural concrete tests in the ⅜ in. top size only. Mixes in which natural sand was used as part of the fine aggregate were made with Slag B. Specimens from these mixes were also utilized in freezing and thawing, thermal coefficient of expansion, linear shrinkage, and thermal conductivity tests. Results of physical tests on Slag B are included in Table 1.

Two additional expanded slags were used in concretes subjected to thermal conductivity tests. These, designated as Slags C and D, had dry loose unit weights in the combined ⅜ in. to 0 size (30 percent ⅜ in. to No. 4 and 70 percent No. 4 to 0 by dry, loose volumes) of 39.6 and 54.2 lb per cu ft, respectively. Similar values for Slags A and B were 59.7 and 49.0 lb per cu ft.

Natural sand fine aggregate

A fine natural sand, physical characteristics for which are recorded in Table 1, was used as part of the fine aggregate in lightweight concrete mixes with Slag B. Table 1 also shows the sieve analyses, by weight, of the slag and sand combinations used. Proportions of 10, 20, and 30 percent sand by dry loose volume of the fine aggregate were used.

Cement

All cement used in the tests came from one lot of Type I portland from a single plant. Air entrainment was obtained by adding a commercial air-entraining agent at the mixer.

Concrete mixing and curing procedures

The aggregates, in a room dry condition, were placed in a tub-type laboratory mixer and a part of the mixing water was added. The mixer was then turned on for a few revolutions to distribute the water through the aggregate. The batch was then allowed to stand for 5 min to permit absorption of water by the aggregates. The cement, remainder of the mixing water, and air-entraining agent were then added, and the mixing continued for 5 min.

The slump and plastic unit weight tests were run at the end of the mixing period and the air content was determined by the volumetric method (ASTM C 173).

Sufficient concrete was mixed in one batch to make all specimens for a given mix design. Specimens were molded, covered with wet burlap, and allowed to remain in the mixing room over night. Molds were then removed and the specimens were stored in a moist room at approximately 72 F and 100 percent relative humidity until tested.

Strength tests

Tests for both compressive and flexural strength were conducted on specimens continuously moist cured until time of test. Procedures followed ASTM methods in general with 6 x 12-in.

cylinders used for compression tests and 3 x 4 x 16-in. beams for flexure. The beams were tested with the 4-in. faces vertical, using third-point loading on a 12-in span. Three specimens were tested from each mix at each test age.

Freezing and thawing tests

The freezing and thawing tests were made on 3 x 4 x 16-in. beams. At the age of 26 days the specimens were removed from the moist room and immersed in water for 48 hr prior to the first freezing cycle. One cycle per day was obtained, freezing in air at −10 F for 20 hr and thawing in water at 40 F for 4 hr. Dynamic moduli of elasticity measurements were made

TABLE 1—PHYSICAL TESTS OF AGGREGATES

Expanded Slag A

	Coarse aggregate			Fine aggregate—	Combined aggregates*— coarse and fine		
	⅜ in.–No. 4	½ in.–No. 4	¾ in.–No. 4	No. 4–0	⅜ in.–0	½ in.–0	¾ in.–0
Sieve analysis, percent passing:							
¾ in.			100.0				100.0
½ in.		100.0	55.0			100.0	81.9
⅜ in.	100.0	60.0	33.0		100.0	87.4	73.0
No. 4	0	0	0	100.0	77.9	68.6	59.7
No. 8				73.0	56.9	50.1	43.6
No. 16				46.8	36.5	32.1	27.9
No. 30				31.0	24.1	21.3	18.5
No. 50				21.2	16.5	14.5	12.7
No. 100				14.2	11.1	9.7	8.5
No. 200				8.6	6.7	5.9	5.1
Fineness modulus	6.00	6.40	6.67	3.14	3.77	4.16	4.56
Weight per cu ft, lb							
Dry loose	41.7	43.2	42.5	63.0	59.7	59.0	58.5
Dry compact	48.8	50.6	49.8	73.6	69.8	66.8	67.0
Absorption, percent	8.0	7.7	7.5	3.6			
Bulk specific gravity, dry	1.6	1.8	1.7	2.3			

*Proportions of coarse to fine aggregate by dry loose volume were: ⅜ in.–0: 30-70; ½ in.–0: 40-60; and ¾ in–0: 50-50.

Expanded Slag B and Natural Sand Aggregates

	Slag coarse aggregate	Slag fine aggregate	Natural sand	Combined aggregates			
				All slag	Slag plus natural sand†		
					10 percent	20 percent	30 percent
Sieve analysis, percent passing:							
¾ in.	100.0		100.0	100.0	100.0	100.0	100.0
No. 4	0	100.0	99.4	77.0	78.2	79.5	80.5
No. 8		77.8	96.4	59.9	63.4	66.6	69.4
No. 16		54.6	84.8	42.0	46.8	51.1	54.8
No. 30		36.6	69.5	28.2	33.0	37.3	41.2
No. 50		22.4	45.0	17.2	20.5	23.5	26.1
No. 100		10.4	9.8	8.0	8.1	8.2	8.2
No. 200		4.2	2.0	3.2	3.0	2.8	2.6
Fineness modulus	6.00	2.98	1.95	3.67	3.51	3.34	3.20
Weight per cu ft, lb.							
Dry loose	35.8	51.2	95.4	49.0			
Dry compact	41.6	58.0	106.4	56.6			
Absorption, percent	13.2	7.1	1.6				
Bulk specific gravity, dry	1.3	1.9	2.56				

†Percentage of natural sand shown is by dry loose volume of fine aggregate only. Coarse and fine aggregates combined on 30-70 basis by dry loose volumes.

periodically in accordance with ASTM C 215. Two specimens were tested to represent each mix.

Thermal coefficient of expansion and drying shrinkage

Specimens 3 x 3 x 19 in., with stainless steel studs molded in the ends for reference points, were used for the coefficient of thermal expansion and drying shrinkage tests. Effective gage length of the beams was 17¼ in. Length measurements were made on a laboratory built comparator consisting of a channel frame with one fixed gage point and an Ames dial at the other end. An Invar reference bar standard was used to compensate for temperature effects on the comparator.

At the age of 5 days the specimens were removed from the moist room and immersed in water at a temperature of 72 F for 5 days. Initial length and weight measurements were then taken. Using single specimens from each mix, the coefficient of thermal expansion was determined for the saturated specimens, obtaining the temperature changes by means of water baths maintained at 35, 72, and 145 F. Linear shrinkage was then measured over a period of 100 days drying in laboratory air. This was followed by oven drying and measurement of the coefficient of expansion of the dry specimens.

Absorption and oven dry unit weight

The oven dry unit weight and absorption values for each mix were obtained from one 6 x 12-in. cylinder. Specimens were immersed in water for 24 hr after 28 days of moist curing. They were then weighed in water and surface dry in air. This was followed by oven drying to obtain the final dry weight.

Thermal conductivity tests

Slabs 1¼ x 12 x 12 in. were used for the thermal conductivity tests. The guarded hot plate method (ASTM C 177) was used. All specimens tested had 28 days moist curing. The slabs were then oven dried and both sides were ground to produce plane surfaces. This was followed by redrying in the oven prior to test.

TEST RESULTS AND DISCUSSION

Strength tests

Compressive strength tests were conducted on all mixes for both insulating and structural concrete after 7 and 28 days and 1 year of continuous moist curing. Seven and 28-day flexural strengths were also determined for each mix using the 3 x 4 x 16-in. beams.

Slag A: ⅜ in. to 0 size—Data on mix proportions and strengths of insulating and structural concretes made with Slag A with ⅜-in. top size material are shown in Table 2. Mixes were included with design cement contents of 4 and 5½ sacks per cu yd for insulating concretes and 7 and 8½ sacks for structural concretes, using a wide range of air contents for each cement content.

Twenty-eight day compressive strengths of the insulating concretes ranged from 560 psi for a 3.92-sack mix with 24 percent air to 1650 psi for a 5.95-sack mix with approximately 10 percent total air. Corresponding oven dry weights of these concretes varied from 82 to 97 lb per cu ft; wet weights of the plastic concretes from 86 to approximately 107 lb per cu ft.

Although some of the structural concrete mixes had high air contents and slumps, all 28-day compressive strengths were over 2000 psi; ranging from 2100 for a 7.41-sack mix with 14 percent air to 3780 for an 8.69-sack mix with 9 percent air. Unit weights for these concretes were, of course, somewhat

higher than for the insulating concrete mixes. Oven dry weights varied from 91.6 to 102.7 lb per cu ft; wet weights from 98.6 to 110.6.

Flexural strengths were quite high compared to the compressive values. For the structural concrete mixes, 28-day flexural strengths varied from 490 to 735 psi, representing 19 to 28 percent of the compressive strengths at the same age. The flexural strength represents an even higher percentage of the compressive strength in the weaker insulating concrete mixes. For these mixes, the flexural strengths of 240 to 465 psi correspond to 28 to 43 percent of the compressive strengths.

The relationships of cement factor, air content, oven dry weight, and 28-day compressive strengths for all concrete in this series are shown graphically in Fig. 1. The insulating concrete mixes required high amounts of air for workability and had correspondingly low weights and strengths. In the structural concrete mixes the effect of air content on strength is more pronounced than for the lean mixes, as shown by the flatter slope of the curves. This shows the importance of keeping air contents as low as is consistent with proper workability for high strength structural concrete made with expanded slag. On

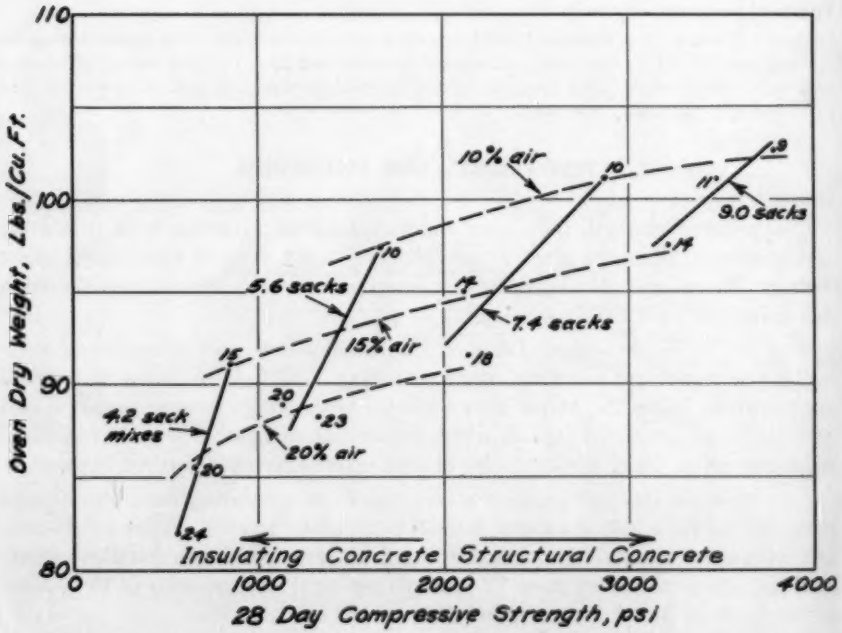

Fig. 1—Relationships of cement factor, air content, weight, and strength of lightweight concrete made with Slag A—⅜-in. to 0 size. (Cement factors shown are averages for the three mixes represented by each solid line. Numbers adjacent to plotted points show air contents of the mixes)

TABLE 2—INSULATING AND STRUCTURAL CONCRETES MADE WITH EXPANDED SLAG A (⅜ IN. TO 0 SIZE)

Design cement content, sacks per cu yd	Insulating concrete						Structural concrete					
	4			5½			7			8½		
Mix No.	1	2	3	4	5	6	7	8	9	10	11	12
Actual cement content, sacks per cu yd	4.46	4.08	3.92	5.95	5.45	5.51	7.59	7.41	7.18	8.69	9.25	9.06
Total water, gal. per cu yd	66.3	60.1	54.3	70.2	57.0	55.1	67.3	61.4	60.3	67.1	65.1	61.3
Dry loose volume of aggregate, cu ft per sack of cement												
Fine	4.70	4.83	4.70	3.62	3.42	3.30	2.75	2.60	2.56	2.29	2.08	2.00
Coarse	2.02	2.07	2.01	1.55	1.47	1.41	1.18	1.11	1.10	0.98	0.89	0.86
Combined	6.37	6.54	6.37	4.90	4.63	4.47	3.72	3.52	3.47	3.11	2.81	2.71
Weight per cu ft of plastic concrete, lb	99.0	91.7	85.6	106.8	92.4	90.6	109.6	102.3	98.6	110.6	109.8	104.7
Air content, percent	14.8	19.5	24.0	10.3	20.0	23.0	9.8	14.3	18.0	8.7	10.9	13.7
Slump, in.	2¾	4½	6¾	1	6	6¼	3¼	6	6½	4½	4¾	6¼
Weight per cu ft oven dry, lb	91.1	85.5	82.0	97.4	88.9	87.2	101.2	95.1	91.6	102.7	101.2	97.6
24-hr absorption, lb per cu ft	15.3	14.8	12.2	15.7	13.2	12.5	13.9	12.8	12.2	13.5	12.8	12.0
Compressive strength,* psi												
7 days	500	430	380	980	720	700	1830	1610	1250	2190	2340	1970
28 days	840	650	560	1650	1120	1260	2860	2100	2130	3780	3480	3210
1 year	1680	1500	1020	2560	2000	1820	4050	3290	2950	4770	4890	4050
Flexural strength,* psi												
7 days	205	160	160	345	285	295	505	460	400	585	640	555
28 days	310	280	240	465	435	400	700	595	490	735	730	725

*Strength test results are averages for three specimens from each mix.

the other hand, lean, high air content mixes may be employed where insulating properties and low weight are the most important characteristics.

A graph of the type shown in Fig. 1 could be used to determine the proper cement factor and air content to produce lightweight concrete having the desired dry weight and compressive strength for a given use, whether insulating or structural grade concrete was required. This particular figure represents a typical slag of ⅜-in. top size and having approximately average unit weight and gradation. Large variations in such factors as unit weight and grading would undoubtedly affect the relationships shown. In such cases, tests to establish a set of curves for the particular grading or unit weight involved would be necessary.

Slag A: ½-in. and ¾-in. to 0 sizes—The data for insulating and structural concrete mixes made with ½-in. and ¾-in. top size expanded slag aggregate are shown in Table 3. Since the air contents and cement factors do not correspond to those for specific mixes with the ⅜-in. size, no direct comparison can be made between Tables 2 and 3. However, the averages for the ⅜-in. slag do match reasonably well the air contents for the mixes with the larger size aggregates. Fig. 2 is a graphical comparison of the test results on the structural concretes for the various top sizes of expanded slag, using average values for the three mixes at each design cement content of the ⅜-in. aggregate series, and the individual mix values for the larger sizes of slag.

Although the number of tests with the ½-in. and ¾-in. aggregates was limited, Fig. 2 shows two definite trends. First, the graph of 28-day com-

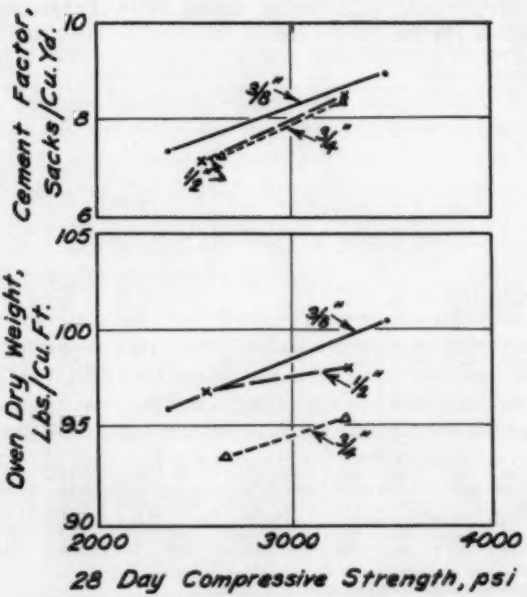

Fig. 2—Effect of aggregate size on dry weight and strength of structural lightweight concrete made with Slag A. (Top size of aggregate as indicated on curves)

pressive strength versus cement factor shows that a given cement factor produces higher strengths—by 200 to 300 psi—with the larger sizes of aggregate. Stated another way, a given strength can be obtained with less cement (0.2 to 0.4 sacks per cu yd) when ½-in. or ¾-in. aggregate is used instead of

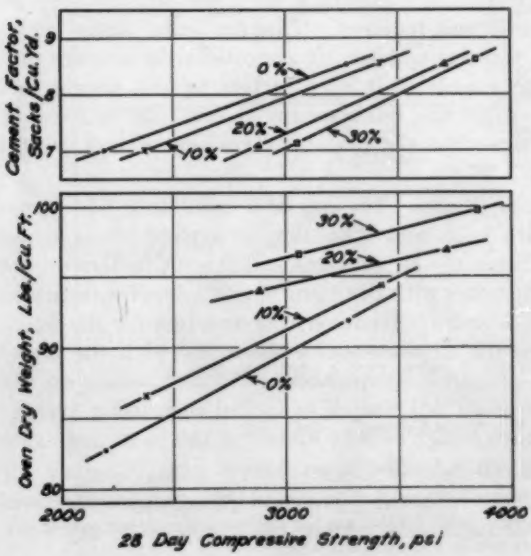

Fig. 3—Effect of natural sand on strength and dry weight of structural lightweight concrete made with Slag B. (Sand contents shown are percentages of the fine aggregate on a loose volume basis)

TABLE 3—INSULATING AND STRUCTURAL CONCRETES MADE WITH EXPANDED SLAG A (½ IN. TO 0 AND ¾ IN. TO 0 SIZE)

	(½ in.–0) Aggregate			(¾ in.–0) Aggregate		
	Insulating concrete	Structural concrete		Insulating concrete	Structural concrete	
Mix No.	13	14	15	16	17	18
Cement content, sacks per cu yd	5.23	7.14	8.55	5.39	7.36	8.51
Total water, gal. per cu yd	60.4	60.5	60.8	52.0	56.5	56.6
Dry loose volume of aggregate, cu ft per sack of cement						
Fine	3.41	2.50	2.00	2.80	2.06	1.73
Coarse	2.32	1.70	1.36	2.80	2.06	1.73
Combined	5.34	3.91	3.13	5.05	3.71	3.12
Weight per cu ft of plastic concrete, lb	97.8	104.5	106.8	93.8	102.2	104.6
Air content, percent	15.9	13.1	12.0	20.2	15.8	13.2
Slump, in.	6	5	5¾	5	5¾	4½
Weight per cu ft oven dry, lb	92.1	96.9	98.1	88.4	93.6	95.6
24-hr absorption, lb per cu ft	14.3	13.0	12.6	11.3	11.1	10.8
Compressive strength,* psi						
7 days	770	1530	1890	830	1530	1980
28 days	1200	2540	3270	1450	2630	3250
1 year	2140	3620	4230	2160	3080	3880
Flexural strength,* psi						
7 days	265	440	595	265	440	540
28 days	400	570	625	390	525	630

*Average values for three tests.

the ⅜-in. top size. Second, the graph of oven dry weight versus strength shows a trend of lower concrete weights at a given strength level when the larger aggregate sizes are used. This is particularly pronounced for the ¾-in. material, and at the higher strengths, where the ¾-in. aggregate produced concrete approximately 4 lb per cu ft lighter than that obtained with ⅜-in. top size material.

Slag B and natural sand—Slag B was used only in structural concrete mixes with 0, 10, 20 and 30 percent natural sand in the fine aggregate (percentages by dry loose volume of the fine aggregate). Results of the tests on these mixes are included in Table 4; the 28-day compressive strengths are shown graphically in Fig. 3.

The top graph in Fig. 3 shows the effects of the natural sand substitutions on strength for the different cement factors, while the lower graph illustrates their effect on oven dry unit weight. The increase in strength with use of natural sand is quite large in the 7-sack per cu yd mixes; somewhat smaller with the 8½ sacks. A mix with 7 sacks of cement and natural sand as 30 percent of the fine aggregate had a strength about the same as an 8-sack mix without sand. Weight increases with natural sand substitutions are larger with lower strength mixes than with high strength mixes. The oven dry unit weights of all concrete made in this series, however, were less than 100 lb per cu ft, while wet weights (of plastic concrete) ranged up to 112 lb per cu ft.

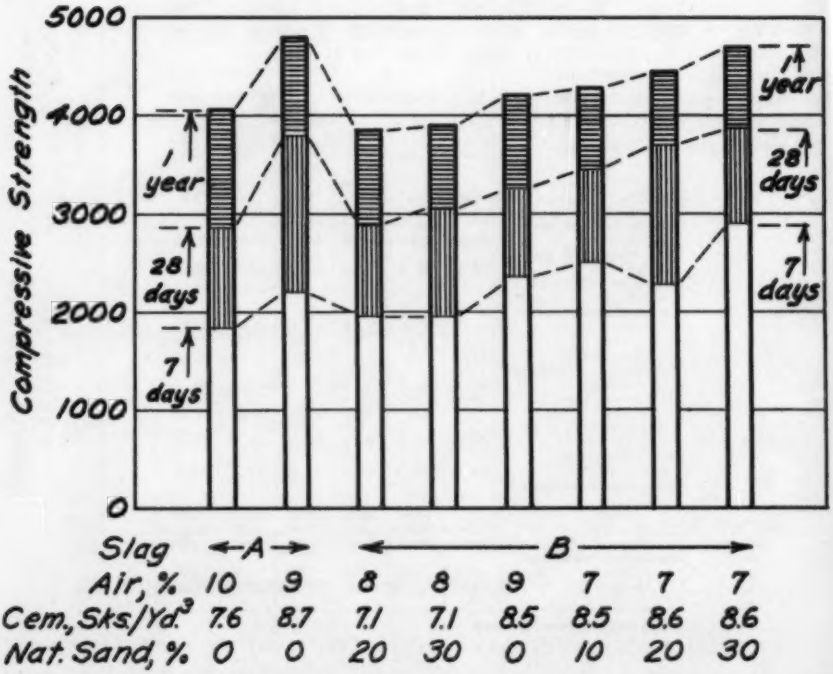

Fig. 4—Compressive strengths of expanded slag structural concretes, ⅜-in. to 0 aggregate and 7-10 percent air. (Natural sand percentages based on loose volume of fine aggregate)

Summary of structural concrete strengths—Data on compressive strengths for the structural concrete mixes with the lower air contents (which would be required for optimum strength designs) are plotted in Fig. 4. Represented in this group are 7.6- and 8.7-sack mixes with expanded Slag A and 7.1- to 8.6-sack mixes with Slag B using varying percentages of natural sand in the fine aggregate. Compressive strengths at 7 and 28 days and 1 year are shown.

Seven-day strengths range from approximately 1800 to nearly 3000 psi, with more than half of the mixes over 2000 psi. Strengths at 28 days for three of the mixes were 3700 psi or higher; six of the eight mixes exceeded 3000 psi. Continued moist curing produced large strength gains between 28 days and 1 year, with many of the mixes testing over 4000 psi at the latter age.

Thermal coefficient of linear expansion

Results of the tests for coefficient of expansion of lightweight structural concretes made with Slag B and varying percentages of natural sand in the

fine aggregate are reported in Table 5. Measured coefficients of expansion varied from 4.8 to 5.2 $\times 10^{-6}$ per deg F for saturated specimens between 35 and 145 F. Values were determined for dry specimens from −8 to 204 F; these range from 4.8 to 5.5 $\times 10^{-6}$.

The data show a tendency toward higher coefficient of expansion values for dry specimens as compared to wet ones, for 8½-sack mixes as compared to 7 sacks, and for mixes with natural sand in comparison to the all slag mixes. However, it is not believed that any of the differences (which are not entirely consistent) are significant, and all values are within the ranges ordinarily found in concrete.

TABLE 4—STRUCTURAL CONCRETE MADE WITH EXPANDED SLAG B AND NATURAL SAND (⅜ IN. TO 0 SIZE)

Natural sand, percent by dry loose volume of fine aggregate	0	10	20	30	0	10	20	30
Mix No.	19	20–21	22–23	24–25	26	27–28	29–30	31–32
Cement content, sacks per cu yd	7.01	7.03	7.10	7.14	8.52	8.54	8.55	8.64
Total water, gal. per cu yd	61.1	63.9	68.1	66.6	65.7	74.1	71.7	68.4
Dry loose volume of aggregate, cu ft per sack of cement								
Fine slag	2.93	2.58	2.44	2.09	2.48	2.18	1.93	1.67
Natural sand	—	0.28	0.60	0.90	—	0.24	0.48	0.72
Coarse slag	1.24	1.23	1.30	1.28	1.07	1.08	1.04	1.02
Combined	3.97	—	—	—	3.38	—	—	—
Weight per cu ft of plastic concrete, lb	93.0	96.9	105.4	108.4	102.1	106.8	109.6	112.2
Air content, percent	14.5	13.7	8.5	8.4	9.0	7.2	7.2	6.6
Slump, in.	6½	5	3½	3	5	2¾	4	3½
Weight per cu ft oven dry, lb	82.7	86.6	94.0	96.6	92.0	94.4	96.9	99.7
24-hr absorption, lb per cu ft	12.6	12.7	13.4	13.0	12.9	13.7	13.8	13.1
Compressive strength,* psi								
7 days	1300	1540	1960	1950	2350	2430	2260	2870
28 days	2200	2380	2870	3040	3260	3430	3700	3860
1 year	3030	2980	3840	3890	4220	4290	4440	4712
Flexural strength,* psi								
7 days	385	420	435	505	505	570	550	605
28 days	500	520	575	615	670	675	685	730

*Average values for three specimens from one mix for 0 percent natural sand, average for six specimens from two mixes for concretes containing natural sands.

TABLE 5—THERMAL COEFFICIENT OF LINEAR EXPANSION OF STRUCTURAL CONCRETES MADE WITH SLAG B AND NATURAL SAND

Mix No.*	Natural sand, percent by volume of fine aggregate	Linear coefficient of thermal expansion, millionths per deg F					
		Saturated specimens			Dry specimens		
		35 to 72 deg	72 to 145 deg	35 to 145 deg	−8 to 140 deg	140 to 204 deg	−8 to 204 deg
7-sack mixes							
19	0	4.4	5.1	4.9	4.6	5.1	4.8
20–21	10	4.4	5.1	4.8	5.1	5.2	5.1
22–23	20	4.5	5.0	4.8	5.4	5.6	5.4
24–25	30	4.8	5.3	5.1	5.2	5.8	5.4
8½-sack mixes							
26	0	4.3	5.2	4.9	5.0	5.6	5.2
27–28	10	4.6	5.3	5.1	5.2	5.6	5.4
29–30	20	4.8	5.1	5.0	5.2	5.6	5.4
31–32	30	5.0	5.4	5.2	5.4	5.8	5.5

*Data on other characteristics of these mixes are given in Table 4.

Freezing and thawing

Data from the freezing and thawing tests on beams from various mixes with both Slags A and B are tabulated in Table 6. The results are shown in terms of the dynamic moduli of elasticity after 300 cycles of freezing and thawing, as percentages of the original values (0 cycles).

Insulating concretes are not subject to durability requirements; nevertheless, several of these were included in the freezing and thawing tests to determine resistance of low cement content mixes to such weathering action. The results show the insulating concretes to have entirely satisfactory durability, with final E values ranging from 78 to 100 percent of the original.

The structural concrete mixes, in all cases, showed no damage from 300 cycles of freezing and thawing. Final E values of 99 to 108 percent of the initial value indicate excellent durability for all of the 12 mix proportions tested.

Drying shrinkage

The effects of laboratory air drying for 100 days on the moisture content and linear shrinkage of structural lightweight concretes made with expanded Slag B are shown graphically in Fig. 5. Although tests were conducted on mixes with 0, 10, 20, and 30 percent natural sand in the fine aggregate, only

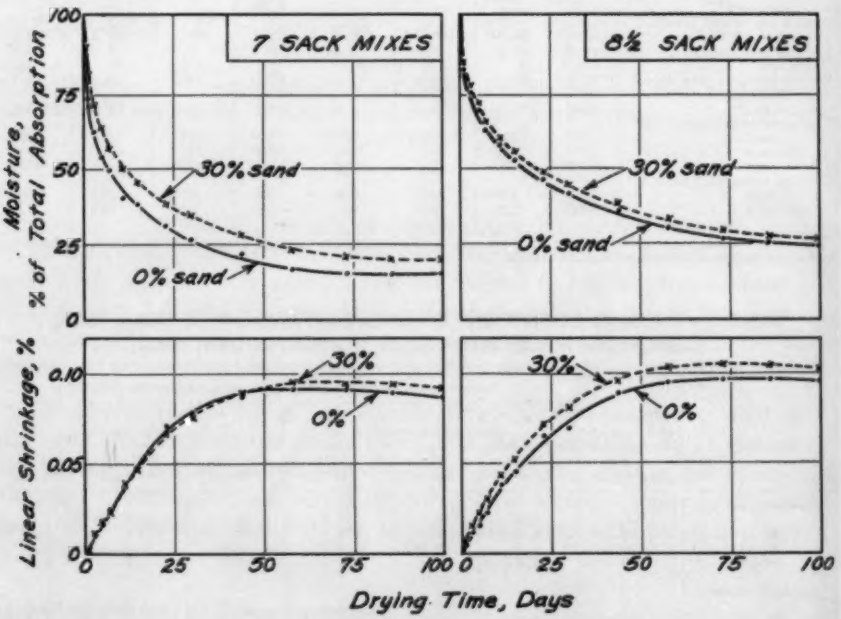

Fig. 5—Shrinkage and moisture content of structural lightweight concrete made with Slag B and exposed to laboratory air drying. (Initial measurements were made after 72 hr immersion in water)

the 0 and 30 percent sand mixes are shown since all curves were quite close together.

For both the 7- and 8½-sack cement contents, the all slag mixes lost moisture a little more rapidly than did the mixes containing natural sand, and the shrinkage was slightly lower. These differences are small and may be caused by the variations in the amounts of water in the mixes (see Table 4), rather than by the use of natural sand. The 8½-sack mixes lost moisture more slowly but had higher shrinkage than the 7-sack mixes, showing the influence of the increased amount of cement.

Only the 8½-sack mix with natural sand had a linear shrinkage value greater than 0.10 percent. It should be noted, however, that the mixes reported here had much higher cement contents than those used for the standard linear shrinkage test in ASTM C 330-53T. If tested with a lower cement content, conforming to the standard, all aggregate combinations would be well within the 0.10 percent shrinkage limit specified in C 330 for structural lightweight concretes. The 7-sack mixes, with less than 0.10 percent shrinkage, had proportions of 1 part of cement to about 4 of aggregate by dry loose volume. Even this is richer than the standard mix for evaluation of the aggregates, which is set at 1:6 by volume in C 330.

Thermal conductivity

Two other expanded slag aggregates were used in addition to Slags A and B in the thermal conductivity tests. Results of all tests are reported in Table 7. A wide range in mixes was employed: four expanded slags; 0 to 30

TABLE 6—CONCRETE CHARACTERISTICS AND RESULTS OF FREEZING AND THAWING TESTS

Mix No.	Natural sand, percent by volume of fine aggregate	Cement factor, sacks per cu yd	Air content, percent	Absorption, lb per cu ft	Dynamic *E* after 300 cycles, percent of original value
Slag A—Insulating Concrete					
1	0	4.46	14.8	15.3	78
3	0	3.92	24.0	12.2	89
4	0	5.95	10.3	15.7	86
6	0	5.51	23.0	12.5	100
Slag A—Structural Concrete					
7	0	7.59	9.8	13.9	102
9	0	7.18	18.0	12.2	99
10	0	8.69	8.7	13.5	101
12	0	9.06	13.7	12.0	104
Slag B—Structural Concrete					
19	0	7.01	14.5	12.6	105
20-21	10	7.03	13.7	12.7	107
22-23	20	7.10	8.5	13.4	104
24-25	30	7.14	8.4	13.0	104
26	0	8.52	9.0	12.9	105
27-28	10	8.54	7.2	13.7	108
29-30	20	8.55	7.2	13.8	107
31-32	30	8.64	6.6	13.1	106

TABLE 7—THERMAL CONDUCTIVITY TESTS ON EXPANDED SLAG CONCRETES
(⅜-in. to 0 Aggregate Except as Noted)

Expanded slag	Natural sand, percent by volume of fine aggregate	Cement factor, sacks per cu yd	Air content, percent	Oven dry weight, lb per cu ft	Thermal conductivity *K*, Btu per hr per sq ft per in. per deg F
A	0	4.08	19.5	87.0	2.37
A	0	5.51	23.0	87.9	2.41
A (¾-in. aggregate)	0	7.14	13.1	98.3	2.91
A (¾ in. aggregate)	0	7.36	15.8	95.2	2.82
A	0	7.41	14.3	96.3	2.74
A	0	8.69	8.7	103.4	3.19
B	0	7.01	14.5	85.7	2.40
B	10	7.00	13.5	88.0	2.61
B	20	7.22	7.4	96.0	3.08
B	30	7.15	8.7	98.8	3.02
C	0	3.92	25.8	60.0	1.50
C	0	5.43	21.4	68.0	1.74
C	0	7.10	10.5	79.8	2.25
C	0	8.41	16.4	73.1	2.17
D	0	5.86	26.4	79.9	2.12
D	0	7.50	25.0	86.0	2.44
D	0	8.85	18.0	97.6	2.83

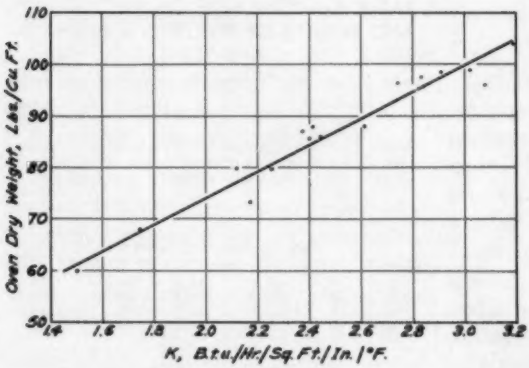

Fig. 6—Relationship of unit weight and thermal conductivity of expanded slag concretes

percent natural sand in the fine aggregate; cement contents from 3.92 to 8.85 sacks per cu yd; and air contents from 7.4 to 26.4 percent. Therefore, the tests cover both insulating and structural concretes with a large range of physical characteristics. Oven dry unit weights for these concretes ranged from 60.0 to 103.4 lb per cu ft and the K values from 1.50 to 3.19.

The relationship of oven dry unit weight and thermal conductivity for these concretes is shown graphically in Fig. 6. The points fall approximately on a straight line, showing that dry unit weight is the principal factor affecting the conductivity. Aggregate characteristics, cement and air contents, and use of natural sand have little influence except as they affect the final unit weight; the conductivity of expanded slag concretes can be predicted with reasonable accuracy from the dry weights. The approximate straight line relationship is typical of results obtained when the comparison is confined to lightweight concrete with a given type of aggregate.

CONCLUSIONS

Based on the tests conducted the following conclusions are drawn:

1. Structural lightweight concretes having 28-day compressive strengths in the range of 2000 to 4000 psi can be readily obtained with both of the typical expanded slags (Slags A and B).

2. The relationships of cement factor, air content, dry weight, and strength for both insulating and structural lightweight concretes with an expanded slag can be incorporated in a single graph. This graph may then be used as a guide in proportioning mixes, with expanded slags comparable in grading and unit weight with the one used in establishing the curves, to meet strength and weight requirements for specific projects.

3. The test results indicate that increasing the top size of the aggregate (from ⅜ to ¾ in.) will result in higher strengths and lower unit weights for a given cement factor.

4. Use of total air contents above 10 percent significantly lowers both the strengths and unit weights of the concretes. Where high strength is a primary factor in the design, air contents should be kept to the minimum consistent with adequate workability.

5. Substitution of natural sand for part of the expanded slag fine aggregate in structural grade concretes resulted in significant increases in strength for a given cement

factor. The unit weights were also increased proportionately. These effects were more pronounced in the leaner mixes.

6. The coefficients of thermal expansion of the lightweight slag structural concretes, with and without natural sand, ranged from 4.8 to 5.5 $\times$ 10^{-6} per deg F. These values correspond closely to the averages usually found in concrete made with gravel, air-cooled slag, or crushed stone aggregates.

7. The expanded slag concretes, in general, had excellent durability in the laboratory freezing and thawing test used. The structural concrete mixes tested were completely unaffected by exposure to 300 cycles of freezing in air and thawing in water.

8. Test results obtained for drying shrinkage appear to be typical of those generally reported for lightweight structural concretes. Small differences obtained by use of natural sand are probably due to variations in the amounts of water in the mixes.

9. Thermal conductivities varied directly with the unit weights of the concretes, with K values from 1.5 to 3.2 for unit weights from 60 to 103 lb per cu ft. These values are comparable to those reported in previous investigations.

ACKNOWLEDGMENT

The investigation was planned by the Technical Committee of the National Slag Association and carried out in the association laboratory under the direction of Fred Hubbard, director of research.

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Discussion of this paper should reach ACI headquarters in triplicate by Feb. 1, 1959, for publication in the June 1959 JOURNAL.

Title No. 55-41

Effect of Axial Compression on Shear Strength of Reinforced Concrete Frame Members*

By J. W. BALDWIN, JR.† and I. M. VIEST‡

An experimental investigation was conducted to determine the effect of axial compression on the shear strength of reinforced concrete members without web reinforcement. It involved tests of knee frames with ratios of axial force to shear varying from 0 to 6, and covered the entire range from failure caused by shear in the absence of axial load to failure caused by eccentric compression. The investigation was an extension of an earlier study involving specimens with the axial load equal to shear and specimens with no axial load, and was thus limited almost entirely to variations of the load ratio.

The observed diagonal tension cracking loads were found in good agreement with the results of the earlier study. On the other hand, the shear compression strength was found to increase with axial load considerably faster than indicated by the earlier tests. A modification of an empirical parameter in an existing theoretical expression for the ultimate strength was found necessary. This modification suggests that at shear compression failure the compatibility of strains, as well as the equilibrium of forces, is a function of the axial load.

INTRODUCTION

An investigation of the shear strength of reinforced concrete frame members without web reinforcement was conducted 1953-55 at the University of Illinois under the sponsorship of the Reinforced Concrete Research Council.[1]§ The investigation included tests of stub beams and symmetrical knee frames with wide ranges of concrete strengths, shear spans, and steel percentages. The knee frames were subjected to an axial load equal to the shear in addition to the shear and bending loads. It was hoped that these tests would indicate the effect of axial load on the shear strength of frame members, but the data were inconclusive; it was decided that further tests were necessary to determine the effect of axial load on shear strength.

Consequently, the investigation reported herein was conducted as an extension of the previous investigation, with the ratio of axial force N to the shear V as the major variable. The N/V ratios ranged from 0 to 6. Since the

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§Reference 1 will hereafter be referred to as the "previous investigation."

specimen with an N/V ratio of 6 failed by typical column failure without the formation of a diagonal tension crack, there was no need for tests of specimens with larger N/V ratios.

Three values of the shear span, shape of the specimen, and minor variations of concrete strength were the additional variables.

Definitions and notations

Critical section is the section of potential shear compression failure. In a knee frame it is the cross section passing through the knee corner. In a stub beam it is the cross section at the edge of the loading stub.

Diagonal tension crack is an inclined crack extending from the longitudinal reinforcement to the critical section and making an angle of approximately 45 deg with the longitudinal reinforcement.

Diagonal tension cracking load is the load at which a diagonal tension crack can be first clearly observed.

Shear compression failure is caused by destruction of the compression zone of the concrete at the critical section above an existing diagonal tension crack.

Test leg is the leg of a knee frame in which failure occurs. It is the leg on which all measurements were made.

a = length of the shear span (Fig. 1-2)

a_c = distance from the section of zero moment (with respect to the centroidal axis) to the starting point of the diagonal tension crack

b = width of cross section

C = compressive force in concrete (Fig. 6)

d = effective depth of reinforcement

E_c = modulus of elasticity of concrete

E_s = modulus of elasticity of steel

f_c' = compressive strength of 6 x 12-in. concrete control cylinders

f_r' = modulus of rupture of concrete

f_s = steel stress at the critical section at shear failure

f_y = yield point stress of reinforcement

h = over-all depth of cross section

$$j_u = 1 - \frac{k_2}{k_1 k_3} \frac{\alpha p f_s}{f_c'}$$ (Fig. 6)

k_1 = coefficient defining the magnitude of compressive force C at failure (Fig. 6)

k_2 = coefficient defining the position of compressive force C in concrete (Fig. 6)

k_3 = ratio of the compressive strength of concrete in flexure to f_c' (Fig. 6)

K = strain reduction coefficient (Fig. 6)

M_{st} = moment around the centroid of tension reinforcement at the critical section

$(M/V)_c$ = ratio of the moment around the centroid of compressive force to the shear at the section of diagonal tension cracking

n = modular ratio

N = axial force

P = load applied to the specimen

P_c = total load on specimen at diagonal tension cracking

P_u = total load on specimen at failure

$$p = \frac{\text{area of tension reinforcement}}{bd}$$

T = tension force in longitudinal reinforcement (Fig. 6)

V = shear

$v_c = \frac{8V_c}{7bd}$ nominal shearing stress at diagonal tension cracking

$v_u = \frac{8V_u}{7bd}$ nominal shearing stress at failure

y = deflection of the end of the shear span in the direction perpendicular to the axis of the knee leg

$$\alpha = 1 + \frac{N}{pbdf_s}$$

ϵ_s = steel strain at shear failure (Fig. 6)

ϵ_u = maximum concrete strain at shear failure (Fig. 6)

SPECIMENS AND TESTS

Specimens

The test series was designed as an extension of the previous investigation. Therefore, a rectangular cross section of 12 x 16 in. was selected with two #6 and four #7 bars as longitudinal reinforcement placed at an effective depth of 14.5 in. Concrete strength of approximately 3500 psi and a yield point for reinforcement of approximately 70 ksi were planned. To obtain shear compression failures, the shear span of the test leg was chosen as 28 in. for ten specimens, and 21 in. and 38 in., respectively, for each of the two others. Two 28-in. specimens were designed with an N/V ratio of 0, and one 28-in. specimen was designed for each of the following N/V ratios: 1/3, 1/2, 2/3, 1, 1½, 2, 3, and 6. The 21- and 38-in. specimens had N/V ratios of 2 (Table 1).

TABLE 1—PROPERTIES OF SPECIMENS

Specimen	Shear span a, in.	Ratio of axial force to shear N/V	Age at testing, days	Concrete strength		Average steel strength	
				f_c', psi	Modulus of rupture, psi	Yield point, ksi	Ultimate strength, ksi
0B28	28	0	60	5450	558	75.3	136.7
0F28	28	0	38	4830	491	45.4	84.7
2F28	28	0.333	50	3980	594	77.1	137.7
3F28	28	0.500	30	3740	481	78.7	138.1
4F28	28	0.667	65	3990	586	75.9	137.3
6F28	28	1.00	64	3080	523	75.3	136.7
9F28	28	1.50	61	3430	470	75.9	137.3
12F28	28	2.00	29	3460	429	75.3	136.7
18F28	28	3.00	57	3770	574	77.1	137.7
36F28	28	6.00	28	3160	435	75.9	137.3
12F21	21	2.00	39	4460	446	45.4	84.7
12F38	38	2.00	39	4190	460	45.4	84.7

All specimens were 12 x 16 in. in cross section. The test legs were reinforced in tension only; effective depth of steel was 14.5 in. and each specimen was reinforced with two #6 and four #7 bars.

The ten specimens with N/V ratios greater than zero were knee frames shown in Fig. 1 and 2. The test leg of each specimen was joined at an angle of 90 deg to another leg of such length as to give the desired N/V ratio. The outer end of each leg was formed into a loading stub to accommodate roller bearing blocks. The test leg had no web reinforcement; the other leg was provided with sufficient web reinforcement to prevent failure in that leg. In addition to the longitudinal and web reinforcement, spacer bars were welded across the ends of the longitudinal reinforcement to provide anchorage, and lifting eyes were embedded at the knee corner and at the end of each leg.

One of the specimens with $N/V = 0$ was a simple stub beam consisting of two test sections, identical with the test leg of the 28-in. knee frames, and a column stub for applying load at the center as shown in Fig. 1. The column stub was reinforced with six #5 vertical bars tied with No. 2 wire ties. Lifting eyes were embedded at each end of the specimen and external stirrups were clamped on the specimen at each support. The specimen was identical to B28B4 of the previous investigation except for the concrete strength and the manner of support; the new specimen was supported on two rollers while B28B4 was supported on knife edges.

The other specimen with $N/V = 0$ was a C-frame consisting of two test sections identical with the test sections of the stub beam and a column section connected at 90 deg to each of

TABLE 2—AVERAGE DIMENSIONS OF BARS

Bar size, No.	Area, sq in.	Deformations		
		Spacing, in.	Height, in.	Gap, in.
Specimens 0F28, 12F21, and 12F38				
6	0.427	0.489	0.050	0.07
7	0.583	0.565	0.060	0.11
All other specimens				
6	0.430	0.468	0.040	0.094
7	0.595	0.582	0.050	0.094

Measurements were made as specified in ASTM Designation A 305-50T, on half of the bars used.

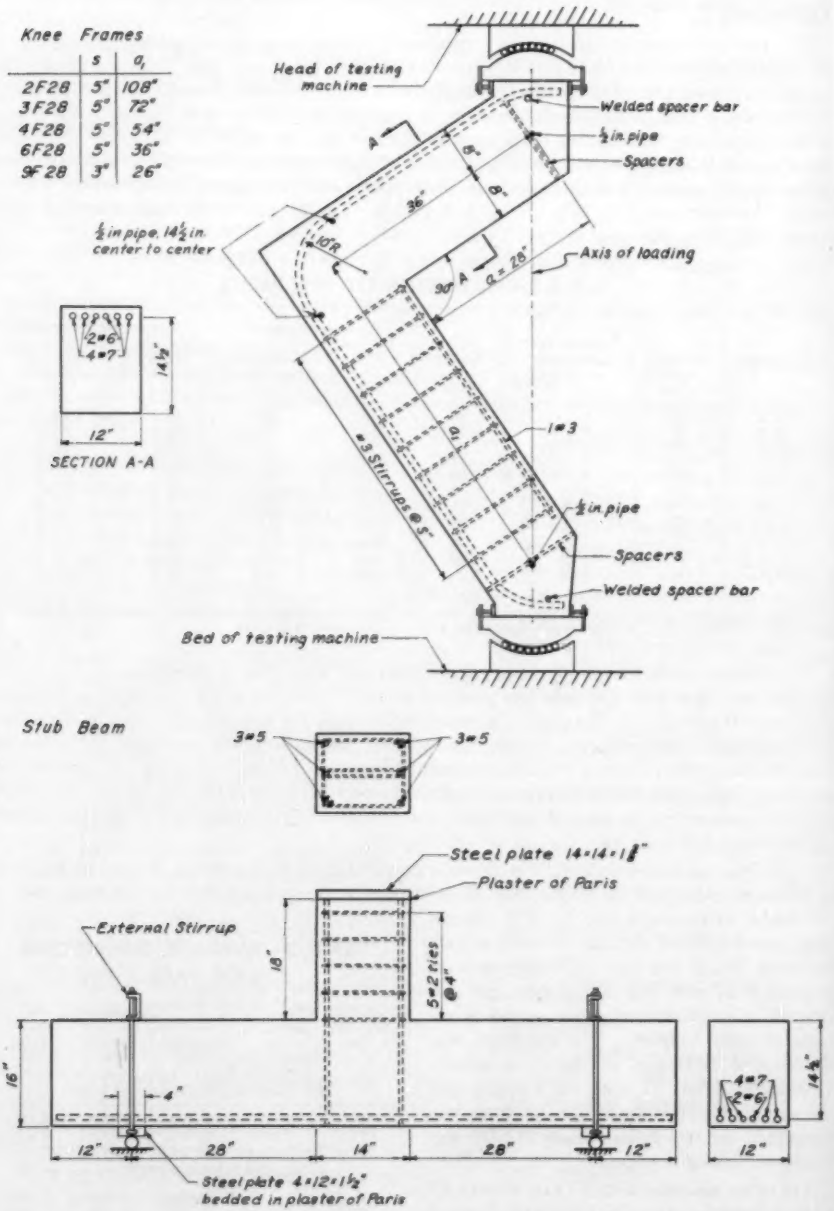


Fig. 1—Details of Specimens I

the test sections as shown in Fig. 2. Except for concrete strength the construction of this specimen was essentially the same as for the knee frames. This specimen may be considered a knee frame with $N/V = 0$. It was tested to determine whether or not the behavior of a stub beam is the same as that of a knee frame with $N/V = 0$.

The specimens are designated by a number, a letter, and a second number. The first number represents the N/V ratio in sixths, the letter is either B or F indicating either a beam or frame, and the second number is the shear span of the specimen in inches. For example, 2F28 is a knee frame with $N/V = 1/3$ and a shear span of 28 in.

Materials

The concrete was made from a standard brand of Type 1 portland cement, Wabash River torpedo sand, and Wabash River gravel. Since there were two sizes of coarse aggregate which varied considerably in gradation from lot to lot, the proportions of the mix were adjusted from time to time to maintain a consistent gradation of coarse aggregate for all batches. The maximum size of the gravel was 1 in. and the fineness modulus of the sand was 3.10.

The average concrete mix was 1:3.97:5.95 by weight, with a cement/water ratio of 1.1 by weight. The concrete was mixed in a nontilting horizontal drum mixer of 6.5-cu ft capacity for approximately 5 min. The compressive strength determined from tests of 6 x 12-in. control cylinders and the modulus of rupture determined from third-point loading of 6 x 6 x 21-in. beams can be found in Table 1.

The reinforcing steel was of hard grade billet steel with deformations perpendicular to the axis of the bar. Dimensions were measured on half the bars of each size (Table 2). The deformations met the requirements of ASTM A 305-53T. The cross sectional area was determined in accordance with ASTM A 15-54T. A coupon was cut from each bar and tested in tension. The bars for each specimen were selected so as to have approximately the same yield point. The average tensile yield point and ultimate strength of the steel for each specimen are listed in Table 1; the average measured modulus of elasticity was 30×10^6 psi.

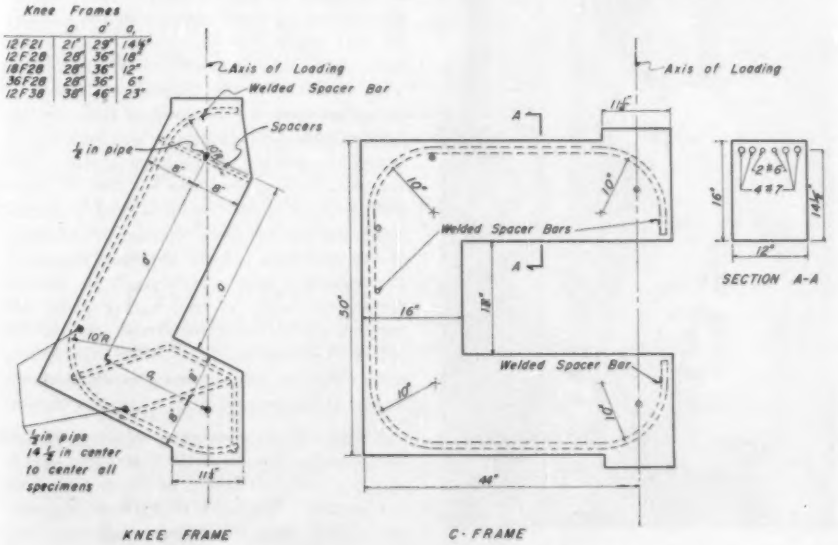


Fig. 2—Details of Specimens II

Fabrication of specimens

The fabrication of specimens was identical to that of the previous investigation except that in addition to the five 6 x 12-in. control cylinders a 6 x 6 x 21-in. modulus of rupture beam was made from each batch of concrete. The control cylinders and beams were cured and tested with the corresponding specimen.

Test equipment and procedure

All knee frames were tested in a Riehle 600,000-lb screw type testing machine, with loads applied in increments of 1000 to 40,000 lb, depending on the expected behavior of the specimen. The testing set-up is shown in Fig. 3. During the first increments of loading, the specimens held the applied load well, but at higher loads it was necessary to operate the machine from time to time to maintain the desired load. At each increment, measurements of steel strain, concrete strain, and deflection were taken.

The steel strains were measured with a 4-in. Berry mechanical strain gage along three gage lines on each side of the frame. The gage lines extended from the critical section toward the end of the test leg. Concrete strains were measured along the center line of the compression surface of the test leg and on one side of the specimen at several locations adjacent to the critical section between the compression surface and the tension steel. Two A-1 and two A-9 SR-4 electric strain gages were used to measure the concrete strain along the center line of the compression surface. The strain distribution on the side of the specimen adjacent to the critical section was measured with nine A-1 SR-4 gages.

Deflections were measured with dial indicators bearing on a deflection bridge as shown in Fig. 3. These deflections were essentially the displacement of the axial load in the direction parallel with the critical section.

The load was applied to the specimens through the roller bearing blocks used in the previous investigation, which allowed free rotation in the plane of the knee frame, but were rigid in the perpendicular plane. The bottom bearing block rested on the bed of the machine while the top bearing block was attached to a spherical seat which was in turn attached to the head of the testing machine. The specimens were inserted in the machine with leather strips between the top loading stub and the bearing block, to distribute the load evenly across the surface of the loading stub. The spherical bearing seat was left free to rotate during the first increment of load to permit compensation for imperfections in the shape of the specimen. After the first increment, the spherical seat was wedged to prevent further rotation. At first, leather strips were inserted also between the bottom loading stub and the bearing block but, after experiencing some difficulty with lateral eccentricities, the leather strips were replaced by plaster of paris.

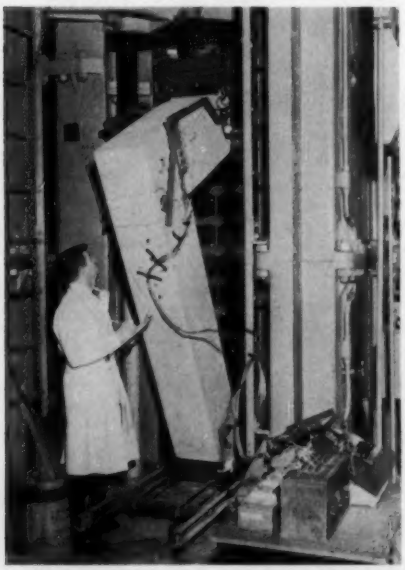


Fig. 3—Typical test set-up (Specimen 3F28)

Except for the manner of support the test procedure for Specimen 0B28 was identical to that for the stub beams of the previous investigation. Specimen 0B28 was supported on 2½-in. diameter rollers while the stub beams of the previous investigation were supported on knife edges.

TEST RESULTS

Test data

The test results are summarized in Table 3. Several specimens after failure are shown in Fig. 4, and various test data are shown in Fig. 5 and 7-11.

Formation of the diagonal tension crack was determined by visual observation. In every case except in 12F21 the crack formed rapidly, extending almost the full distance from the steel to the critical section; thus the instant of crack formation was well defined. The corresponding loads are the diagonal tension cracking loads listed in Table 3. The diagonal tension crack in Specimen 12F21 developed slowly over a range of about 20 kips. Thus the diagonal tension cracking load was not well defined and the load listed in the table may be somewhat too high.

The nominal shearing stress v_c was computed as $8V/7bd$, where V is the shear caused by P_c and the dead load at the intersection of the diagonal tension crack with the steel. The term v_c/f_c' listed in Table 3 is the nominal shearing stress at diagonal tension cracking divided by the compressive strength of the 6 x 12-in. control cylinders.

The failure load P_u is the maximum load applied to the specimen during the test. The nominal shearing stress v_u was computed from P_u as $8V/7bd$. Dead load shear was not included in v_u because it was always less than 2 percent of the applied load. The term v_u/f_c' listed in the table was computed as the nominal shearing stress at failure divided by the compressive strength of the 6 x 12-in. control cylinders.

All specimens except 36F28, 18F28, and 12F21 failed in shear compression. Specimen 36F28 failed in flexural compression. The failures of Specimens 18F28 and 12F21 had the characteristics of both shear compression and flexural compression failures.

The maximum deflections listed in Table 3 are the displacement of the axial load in the direction parallel to the critical section. Whenever possible, one member of the testing staff watched the deflection at the instant of failure. When this was not possible it was necessary to extrapolate the load-deflection curve to obtain the deflection at failure. Since the load-deflection curve becomes flat near failure, this procedure was somewhat less accurate than the direct readings.

The maximum steel strains recorded in Table 3 are the averages of the readings on the two gage lines at the critical section. Since it was impossible to obtain a steel strain reading at the instant of failure, all ultimate steel strain values are extrapolated.

The maximum concrete strains were difficult to obtain, because at loads near failure extensive crushing of the concrete was present at the critical section. The values listed in Table 3 are the maximum measured values.

The nominal shearing stress at diagonal tension cracking of 9F28 was low as compared with the specimens on either side of it in the test series. Deflections, steel strains, and crack patterns indicated that there was some lateral eccentricity in the loading of this specimen. Several attempts were made to

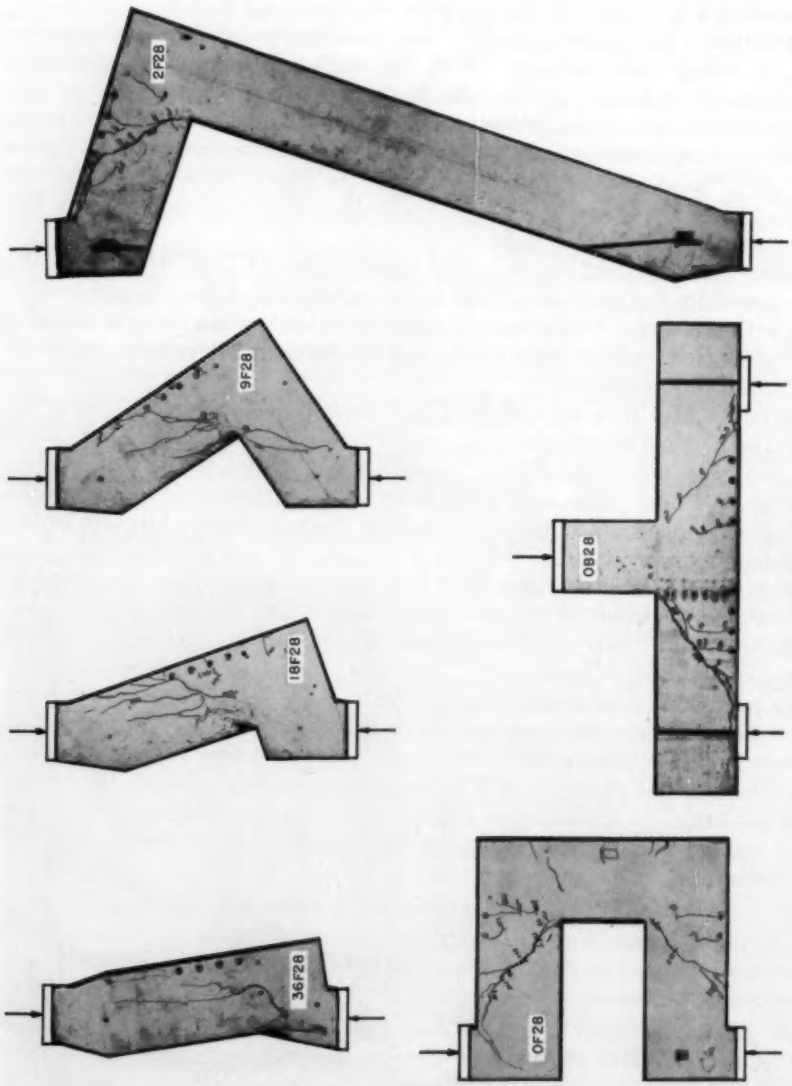


Fig. 4—Specimens after failure

TABLE 3—TEST RESULTS

Specimen	Diagonal tension cracking			Failure				Maximum strain in millionths	
	Load P_c, kips	Shear stress v_c*, psi	$\frac{v_c}{f_c'}$*	Load P_u, kips	$\frac{v_u}{f_c'}$	Mode†	Maximum deflection, in.	Steel ϵ_s	Concrete ϵ_c
0B28	76	251	0.046	130	0.078	S	0.21	1200	3010
0F28	40	263	0.055	61	0.083	S	0.35	1360	6150
2F28	44	254	0.064	74	0.116	S	0.43	1330	4980
3F28	48	270	0.072	93	0.146	S	0.42	1660	3880
4F28	60	315	0.079	79	0.109	S	0.25	1100	4830
6F28	59	265	0.086	90	0.135	S	0.29	1280	3330
9F28	65	232	0.068	128	0.135	S	0.34	1160	6550
12F28	119	345	0.100	190	0.161	S	0.37	1120	5590
18F28	196	404	0.107	234	0.129	S–F	0.32	830	7630
36F28	—	—	—	359	0.123	F	0.11	550	7200
12F21	191	560	0.125	240	0.158	S–F	0.18	1020	7100
12F28	100	294	0.070	112	0.078	S	0.37	950	—

*Includes dead load shear.
†S—Shear compression failure.
F—Flexural compression failure.

correct this condition by unloading the specimen and adjusting the spherical seat. The diagonal tension crack formed during one of the reloadings at a load lower than that which had been applied to the specimen prior to unloading. Thus it is not known whether the lower cracking load was caused by the eccentricity or by the repetitions of loading.

Behavior of specimens under load

All specimens behaved similarly during the first few increments of loading. The first tension cracks formed at the section of maximum moment; with increasing load additional cracks formed farther away from this section. The extent of tension cracking varied inversely with the N/V ratio, as would be expected from the fact that with increasing N/V ratio not only the moment for a given load decreases, but also the neutral axis moves toward the tension side of the section. The cracking patterns for several specimens are shown in Fig. 4.

In all specimens except 12F21, 12F28, 18F28, and 36F28, which were the specimens with the highest N/V ratios, some of the tension cracks began curving over toward the section of maximum moment as loading progressed. At a somewhat higher load a diagonal tension crack formed, either through or close to the compression end* of one of the inclined cracks; the diagonal tension crack intersected the tension steel between 12 and 18 in. from the critical section, and its angle with the axis of the test leg varied from approximately 30 to 45 deg. Specimens 12F21, 12F28, and 18F28 formed diagonal tension cracks similar to those formed in the first six specimens except that the cracks were somewhat less inclined which resulted in a deeper compression zone above the crack.

During the interval between diagonal tension cracking and failure the only changes in the appearance of the specimens were increasing deflections, widen-

*The end of a crack which is nearest the compression surface of the specimen is referred to as the compression end of the crack.

ing of the diagonal tension cracks, and local crushing of the compression zones at loads near failure. The extent of crushing observed before failure varied directly with N/V. There was no crushing before failure in Specimen 0B28; crushing was observed at a load 10 kips below failure in Specimen 2F28 but at a load of 50 kips below failure in Specimen 36F28.

All specimens failed by destruction of the compression zone of concrete, with the steel stresses always below the yield point value. The failure of Specimen 0B28 can best be described as violent. It was sudden and complete, occurring before any local crushing was evident and was accompanied by splitting along the steel to the end of the beam. This failure was of the typical shear compression type (Fig. 4). Each specimen with a successively higher N/V ratio failed less violently with more local crushing in the compression zone before the final failure. Nevertheless, all failures were of the shear compression type except in Specimens 18F28 and 12F21 which seemed to be transitions between shear compression and flexural compression failures. There was a general crushing of a deep compression zone with longitudinal cracks forming throughout the entire length of the specimen. The failure of 36F28 (Fig. 4) was of the typical flexural compression type found in eccentrically loaded columns. There was no diagonal tension crack and the failure was accompanied by longitudinal cracking and general crushing.

ANALYSES

Diagonal tension cracking

The following expression for predicting the nominal shearing stress at diagonal tension cracking was developed from the previous investigation:

$$v_c = \left[1.425 + \frac{38}{\frac{(M/V)_c}{npd} + 10} \right] \sqrt{f_c'} \quad \ldots\ldots (1)$$

For a combination of moment, shear, and axial load

$$(M/V)_c = a_c - \frac{N}{V}\left(\frac{h}{2} - \frac{d}{8}\right) \quad \ldots\ldots (2)$$

The distance a_c depends on the a/d ratio and is given by the following expression:

$$a_c = 0 \text{ for } d \geqq a$$

$$a_c = a - d \text{ for } a \geqq d \geqq \frac{a}{2}$$

$$a_c = \frac{a}{2} \text{ for } \frac{a}{2} \geqq d$$

Ten of the specimens reported herein have only the N/V ratio and f_c' as variables. Eq. (1) indicates that v_c varies approximately with $\sqrt{f_c'}$. There-

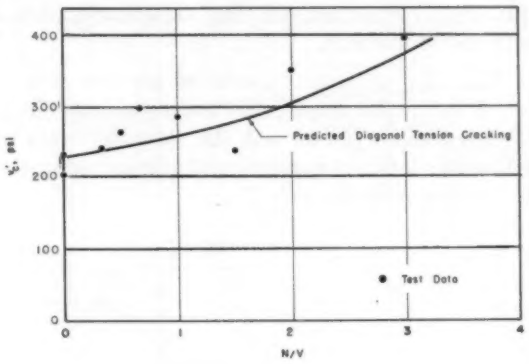


Fig. 5—Effect of N/V ratio on diagonal tension cracking load

fore, multiplying the values of v_c by $\sqrt{3520/f_c'}$ corrects the test data to the average f_c' of 3520 psi. Plotting the corrected values—$v_c' = v_c\sqrt{3520/f_c'}$—against the only remaining variable, N/V, shows the effect of N/V on the diagonal tension cracking load. This has been done in Fig. 5.

It should be noted that the solid line in Fig. 5 represents an analysis developed earlier on the basis of tests of more than 70 specimens. Most of the test values from this investigation, shown in Fig. 5 as dots, fall above the solid line. However, the differences were considered too small and the data too few to justify an attempt at deriving a new analytical expression.

The ratios of v_c *test*$/v_c$ *calc* calculated from Eq. (1) and (2) are listed in Table 4. The average value of v_c *test*$/v_c$ *calc* is 1.07 and the standard deviation is 0.111. It appears, therefore, that Eq. (1) is applicable for a wide range of N/V ratios.

Shear compression—Previous analysis

If a diagonal tension crack forms without causing failure of the member and if flexural failure does not occur, the member fails in shear compression. For this type of failure the maximum moment about the steel at the critical section is defined as the shear moment capacity. By assuming a strain distribution as shown in Fig. 6, the following expression for predicting the shear moment capacity was developed from the previous investigation:

$$\frac{M_{st}}{f_c' bd^2} = \frac{\alpha p f_s}{f_c'}\left(1 - \frac{k_2}{k_1 k_3}\frac{\alpha p f_s}{f_c'}\right) \quad \ldots\ldots(3)$$

where

$$\alpha = 1 + \frac{N}{pbdf_s} \quad \ldots\ldots(4)$$

$$f_s = \tfrac{1}{2} E_s K\epsilon_u\left[-1 + \sqrt{1 + \frac{4k_1 k_3 f_c'}{\alpha p E_s K\epsilon_u}}\right] \quad \ldots\ldots(5)$$

Eq. (3)-(5) were derived from statics and from the assumed strain distribution shown in Fig. 6.

The only unknown quantities are the parameters k_1k_3, k_2/k_1k_3, and $K\epsilon_u$ which define the shape of the concrete stress block, the position of the centroid of compressive forces, and the concrete strain at failure, respectively. The following empirical expressions for these parameters were determined from the previous investigations:

$$k_1 k_3 = \frac{800 + f_c'}{70 + f_c'} \quad \text{.....(6)}$$

$$\frac{k_2}{k_1 k_3} = 0.44 \quad \text{.....(7)}$$

$$10^4 K\epsilon_u = \frac{1.116\frac{a}{d} + 0.174}{\frac{a}{d} - 0.872} \geqq 3\frac{a}{d} \text{ (always a positive quantity)} \quad \text{.....(8)}$$

It can be seen from Eq. (3)-(8) and from Fig. 6 that N enters the analysis only through the equations of statical equilibrium. Thus this analysis as-

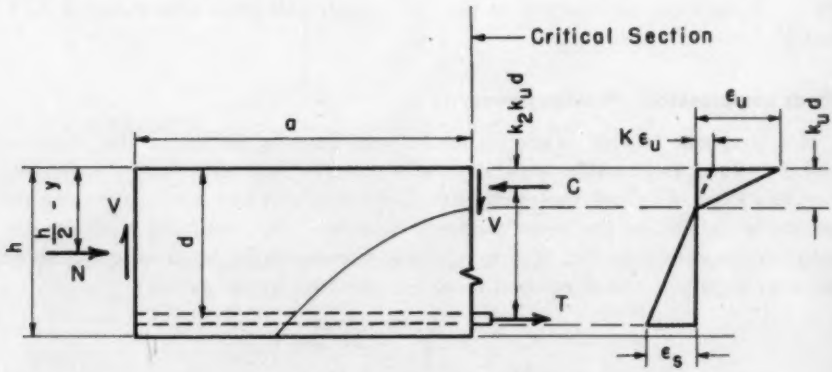


Fig. 6—Assumed strain distribution at shear compression failure

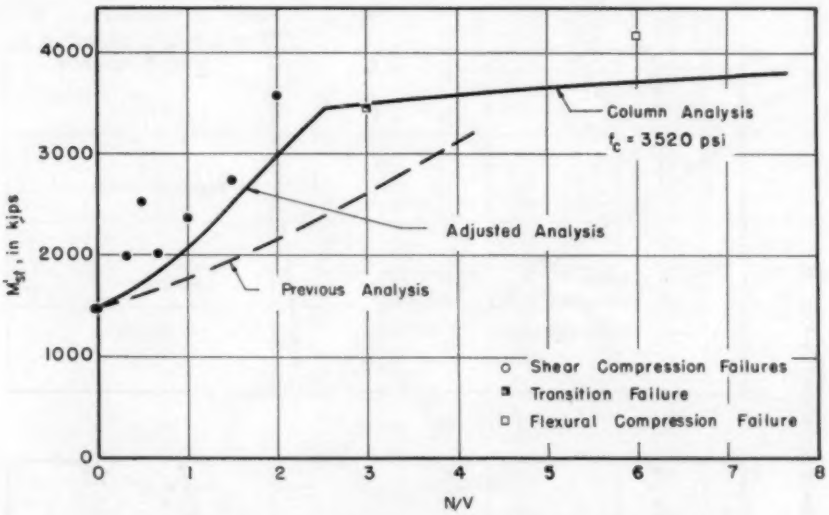


Fig. 7—Effect of N/V ratio on ultimate shear moment

sumes that the axial compression affects the shear compression strength only insofar as it changes the conditions of statics.

Eq. (3) and (5) indicate that M_{st} varies approximately with the square root of f_c'. Using this relationship, M_{st} for ten of the specimens of this investigation may be corrected for small variations in concrete strength and then plotted against N/V, the only other variable. The test data thus corrected to the average concrete strength of 3520 psi as $M'_{st} = M_{st}\sqrt{3520/f_c'}$ are shown in Fig. 7. The dashed line in Fig. 7 represents the solution of Eq. (3)-(8) for $f_c' = 3520$ psi. It is evident from this plot that the axial compression had some effect on the shear moment capacity beyond that accounted for by the equations of statics. Furthermore, this additional effect increased with increasing N/V ratio.

Adjustment of shear compression analysis

Both the data reported herein and those of the previous investigation were examined to determine the reason for the discrepancies between the test results and the existing analysis. Fig. 8a indicates that the assumption regarding the strain distribution is probably fairly accurate. This typical strain distribution plot shows that there is a definite break in the strain distribution line after diagonal tension crack formation. The break in the line after diagonal tension cracking is believed to be primarily due to concentrated rotations at the head of the crack as is assumed in the analysis. However, some nonlinearity was observed also before diagonal tension cracking; this is better illustrated in Fig. 8b which shows the strain distribution for Specimen 36F28 in which no diagonal tension crack formed; this nonlinearity was

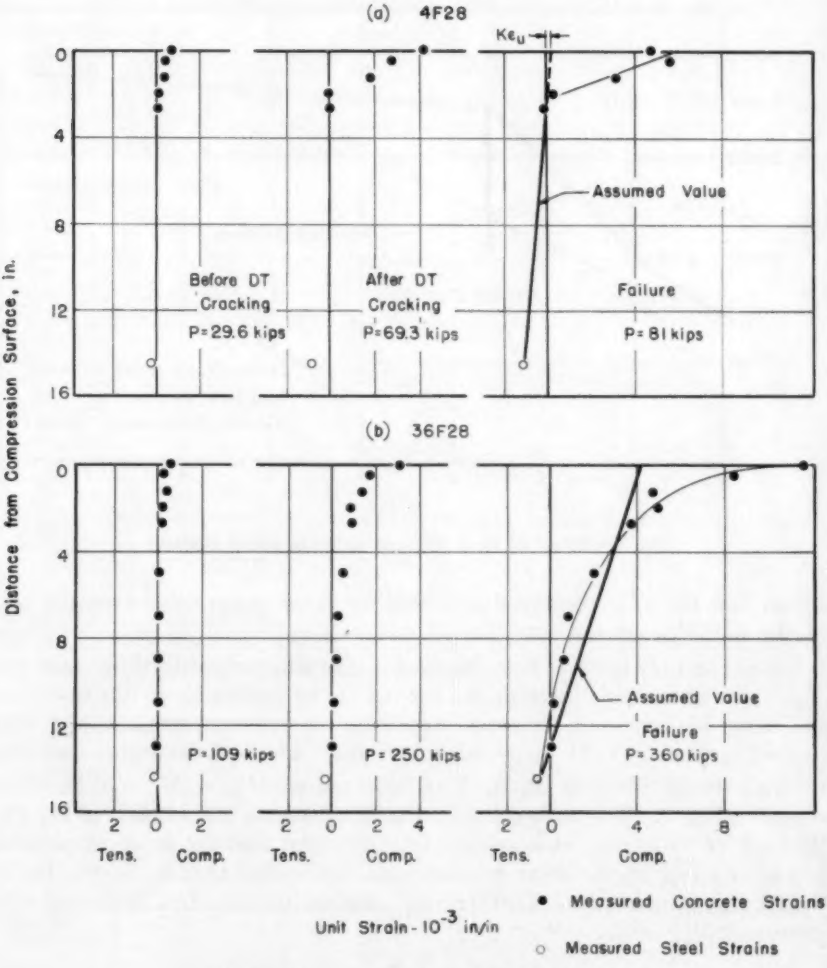


Fig. 8—Measured strains at critical section

caused by the proximity of the re-entrant knee corner and the accompanying stress concentrations.

The parameters k_2/k_1k_3, k_1k_3, and $K\epsilon_u$ were determined empirically from the previous investigation as given in Eq. (6)-(8). Parameter $k_2/k_1k_3 = 0.44$ was found to be representative of all shear compression failures. Fig. 9, in which all values are measured quantities, indicates that this value agrees reasonably well with the results of both investigations.

The parameter k_1k_3 was found to vary with f_c' [Eq. (6)]. Since there was little variation in f_c' in this investigation and no indication was found that

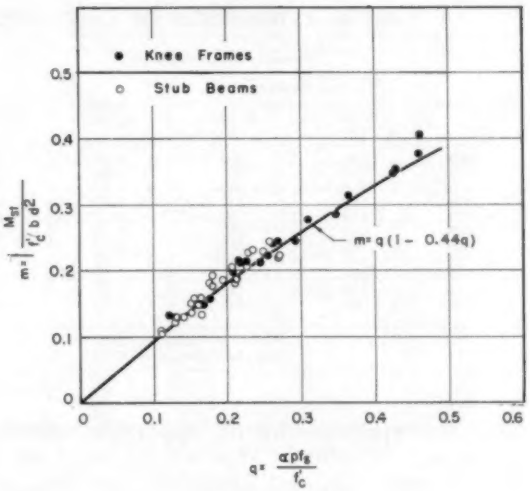


Fig. 9—Correlation of measured quantities at shear compression failure

this parameter is a function of any other variables, Eq. (6) was accepted as being correct.

On the other hand, the data from this investigation indicate that $K\epsilon_u$ is a function of N/V in addition to the a/d ratio suggested by Eq. (8).

For beams N/V is equal to zero. It can be assumed, therefore, that for beams the parameter $K\epsilon_u$ is a function of a/d only. The test values of $K\epsilon_u$ for 0B28 and for stub beams tested in the previous investigation were plotted against a/d. A curve fitting the data gave the following expression for $10^4\ K\epsilon_u$ when $N/V = 0$:

$$\frac{0.9\,\frac{a}{d} + 0.1}{1.0\,\frac{a}{d} - 1.0} \geqq 3\,\frac{a}{d} \text{ (always a positive quantity)}$$

This expression gives almost the same values as Eq. (8).

Assuming that the contribution of N to $K\epsilon_u$ is additive to the effect of a/d, an expression was derived from the data for both symmetrical and unsymmetrical knee frames from this investigation and the previous investigation.

The following expression was found for the contribution of axial compression to $K\epsilon_u$:

$$3.5\left(\frac{d}{a}\,\frac{N}{V}\right)^2 10^{-4}$$

TABLE 4—COMPARISON OF CALCULATED AND TEST RESULTS

Specimen	Diagonal tension cracking			Failure		
	Nominal shearing stress					
	v_u *test*, psi	v_u *calc*, psi	$\frac{v_u \text{ test}}{v_u \text{ calc}}$	M_u *test*, in.-kips	M_u *calc*, in.-kips	$\frac{M_u \text{ test}}{M_u \text{ calc}}$
0B28	251	268	0.94	1820	1830	1.00
0F28	263	256	1.03	1710	1710	1.00
2F28	254	249	1.02	2120	1700	1.25
3F28	270	247	1.09	2610	1750	1.50
4F28	315	258	1.22	2140	1930	1.11
6F28	265	242	1.10	2210	1910	1.16
9F28	232	273	0.85	2700	2470	1.09
12F28	345	295	1.17	3540	2950	1.20
18F28	404	387	1.04	3580	3980	0.90
12F21	560	449	1.25	3690	4110	0.90
12F38	294	280	1.05	2560	2310	1.11
		Average: 1.07 Standard deviation: 0.111			Average: 1.11 Standard deviation: 0.164	

Thus the expressions for $K\epsilon_u$ applicable both to beams and knee frames are as follows:

for $\frac{a}{d} \geqq 1.32$

$$10^4 K\epsilon_u = \frac{0.9\frac{a}{d} + 0.1}{1.0\frac{a}{d} - 1.0} + 3.5\left(\frac{d}{a}\frac{N}{V}\right)^2 \quad \text{.....(9a)}$$

and for $\frac{a}{d} \leqq 1.32$

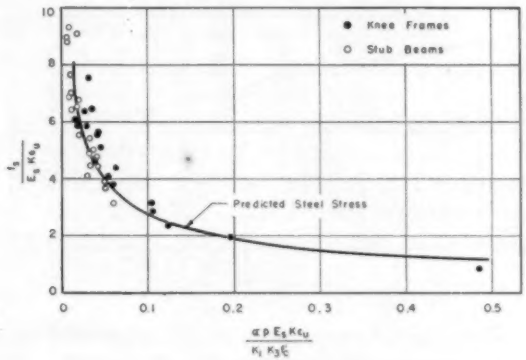
$$10^4 K\epsilon_u = 3\frac{a}{d} + 3.5\left(\frac{d}{a}\frac{N}{V}\right)^2 \quad \text{.....(9b)}$$

Fig. 10 and 11 were made as a check of the analysis. The dots in Fig. 10 represent the measured values of steel stress from both investigations; the solid line represents Eq. (5).* The test points shown in Fig. 11 are the measured distances from the compression surface to the compression end of the diagonal tension crack in this investigation; the solid line represents the depth of neutral axis as predicted by Eq. (4) and (5) equivalent to the depth of the crack since it is assumed in the analysis that the depth of the neutral axis and the depth of the diagonal tension crack are identical. The correlations shown in the two figures, particularly that of the crack depth, indicate that the analysis is reasonably accurate.

*All analyses were performed by successive approximation described in Reference 6. The following additional equation of statics is needed for this method:

$$\alpha = 1 + \frac{j_u}{\frac{V}{N}\frac{a}{d} - \frac{h}{2d} + \frac{y}{d} + 1 - j_u}$$

Fig. 10—Steel stress at shear failure



The test data of this investigation are compared with the adjusted analysis in Fig. 7 and Table 4. The solid line in Fig. 7 represents the solution of Eq. (3)-(7) and (9) for $f_c' = 3520$ psi. It should be noted that Eq. (9) was developed from the data of both this investigation and the previous investigation. In general, the test values were somewhat lower for the previous investigation than for this investigation, which resulted in test points falling above the solid line in Fig. 7. Nevertheless, the points follow the trend indicated by the "adjusted" analysis. The ratios of test to calculated values for the shear moment are listed in Table 4. The average of these values for 11 specimens of this investigation is 1.11 and the standard deviation is 0.164.

Table 5 compares the adjusted analysis with test data of the previous investigation. It can be seen that the agreement is good. For stub beams, the average value for M_{test}/M_{calc} of 1.03 for the adjusted analysis is close to the value of 1.01 for the previous analysis.[1] This is as expected because for members without axial load the two analyses differ only by a small change in $K\epsilon_u$. For the knee frames, the adjusted analysis gives an average value of M_{test}/M_{calc} of 0.96 compared to a value of 1.11 given by the previous analysis. It appears that including the N/V term in the equation for $K\epsilon_u$ improved the agreement between the test data and the analysis.

The average value of M_{test}/M_{calc} for all knee frames, including this and the previous investigation, is 1.03 and the standard deviation is 0.134.

TABLE 5—COMPARISON OF ADJUSTED ULTIMATE MOMENT EXPRESSIONS WITH RESULTS OF PREVIOUS INVESTIGATION

Stub beams	$\frac{M_{test}}{M_{calc}}$	Knee frames	$\frac{M_{test}}{M_{calc}}$
B14B2	0.89	F21B2	0.88
E2	1.14	B2R	0.99
A4	0.91	B4	0.83
B4	0.91	B4R	0.93
E4	1.05	C4R	1.00
A6	0.96	D4	1.13
B6	1.05	A6	0.97
B21B2	0.88	B6	1.10
A4	1.15	F38A6	0.93
B4	1.03	B6	0.86
A6	0.94		
B6	1.10	Average:	0.96
B28B2	1.18	Standard deviation:	0.101
E2	1.23		
A4	1.20		
B4	0.98		
E4	1.21		
A6	0.85		
B6	1.06		
B40B4	0.94		
Average:	1.03		
Standard deviation:	0.12		

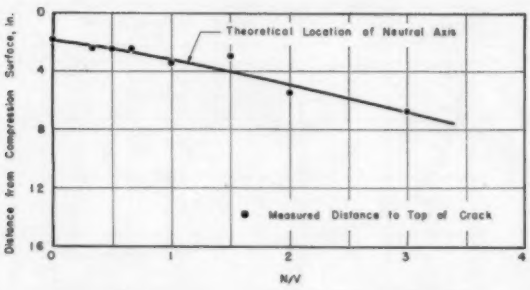


Fig. 11—Depth of crack

The only difference between the adjusted analysis and the previous analysis is the assumption regarding the effect of N. The previous analysis assumed that the only effect of N was that accounted for by the equations of statics. The adjusted analysis assumes that N affects the compatibility of strains as well as the equations of statics.

Flexural compression failures

Specimen 36F28 failed in flexural compression. Hognestad[2] developed equations for predicting the flexural compression strength of eccentrically loaded columns.* The strength of 36F28 was found to be 17 percent higher than that predicted by Hognestad's equation. It is believed that this high value was probably caused by the restraint offered the concrete by the knee corner.

In Fig. 7 the upper solid line represents the failure loads for the specimens of this investigation computed from Hognestad's equations using $f_c' = 3520$ psi. The intersection of this line with the line representing Eq. (3) is the theoretical transition point between shear compression and flexural compression failures.

It can be seen from Fig. 7 that Specimen 18F28 is close to the point of transition. This is in agreement with the test observations; the specimen appeared to fail by a mixture of shear compression and flexural compression failures. The ratio of observed ultimate load to the calculated flexural compression load was 0.96.

SUPPLEMENTARY FINDINGS

Effect of knife edge supports

Specimen 0B28 was tested to compare the knife edge supports used in the previous investigation with the usual roller supports. The diagonal tension cracking load of 0B28 corrected for concrete strength was 5 percent lower than that of B28B4, and the corrected failure load was 1 percent higher. Since both of these percentages are well within the range of the normal scatter, it appears that the knife edges probably had no significant influence on either the diagonal tension cracking load or the shear moment capacity.

*Since the specimens of this investigation were cast horizontally $k_3 = 1$ was used instead of the 0.85 suggested by Hognestad.

Comparison of stub beams with C-frames

Specimen 0F28, identical with Specimens B28B4 and 0B28 except for concrete strength and shape, was tested to compare the behavior of a stub beam to that of a knee frame with $N/V = 0$. The corrected value of diagonal tension cracking load for 0F28 was 9 percent higher than that of 0B28, and 4 percent higher than that of B28B4. The corrected value of ultimate load was within 1 percent of that of both stub beams. Since these differences are also within the range of normal scatter, it appears satisfactory to consider stub beams as knee frames with N/V equal to zero.

SUMMARY

(1) Twelve specimens were tested to investigate the effect of axial compression on the shear strength of reinforced concrete members without web reinforcement. The test sections of all specimens were similar. The ratio of the axial force to shear was the major variable and concrete strength, a/d ratio, and shape of specimen were the minor variables. Eleven specimens developed diagonal tension cracks; after some increase of load, nine of these failed in shear compression and two by a combination of shear compression and flexural compression. The twelfth specimen failed in flexural compression without the formation of a diagonal tension crack.

(2) The diagonal tension cracking loads were found to be in good agreement with the analytical expression derived in a previous investigation.[1]

(3) The shear moment capacities were compared with the equations of a previous analysis[1] and were found considerably in excess of the computed values; the discrepancy increased with increasing N/V ratio. It appears that the axial compression affects not only the conditions of statics, but also the compatibility of strains.

(4) The previous analysis[1] was modified so as to predict the shear moment capacity of knee frames and stub beams with reasonable accuracy.

(5) The moment at flexural compression failure was compared with the equations developed by Hognestad[2] for eccentrically loaded columns. Although the test value was larger than the computed one, Hognestad's analysis appears to predict the flexural compressive strength with reasonable accuracy.

ACKNOWLEDGMENTS

The investigation was carried out during 1955-56 at the Talbot Laboratory of the University of Illinois as a part of the regular research program of the Department of Theoretical and Applied Mechanics.

Considerable advice on fabrication of specimens and testing procedure was given by Dr. J. Morrow, assistant professor of theoretical and applied mechanics, and able assistance was rendered by laboratory mechanics M. R. Penny and E. Bryant.

Reinforcing steel for the specimens was donated by the Laclede Steel Co.

REFERENCES

1. Morrow, JoDean and Viest, I. M., "Shear Strength of Reinforced Concrete Frame Members Without Web Reinforcement," ACI JOURNAL, V. 28, No. 9, Mar. 1957 (*Proceedings* V. 53), pp. 833-869.

2. Hognestad, Eivind, "A Study of Combined Bending and Axial Load in Reinforced Concrete Members," *Bulletin* No. 399, Engineering Experiment Station, University of Illinois, 1951, 128 pp.; published also as *Bulletin* No. 1 of the Reinforced Concrete Research Council of the Engineering Foundation.

Discussion of this paper should reach ACI headquarters in triplicate by Feb. 1, 1959, for publication in the June 1959 JOURNAL.

CONCRETE BRIEFS

Notes from Field and Office

Particle Shape of Fine Aggregate Affects Water Requirement for Mortars 655
Concrete's Etymological Offspring 656
Problems and Practices 657

Particle Shape of Fine Aggregate Affects Water Requirement for Mortars

By EDWIN N. LORENZEN*

In testing the compressive strength of mortars containing sands of questionable structural strength, the preparation of the sands to meet a common grading will eliminate to the greatest extent possible variations caused by differences in grading. This was done for the series of tests described herein, but it was found that for a given flow and a water-cement ratio of 0.60 by weight, the amount of sand added to a cement-water paste varied with different sands even though the *gradings* were identical.

Five different sands were obtained from various sources, ranging from 100 percent manufactured to a natural rounded beach sand. Each sample of sand was separated by sieving and recombined to the grading shown in Table 1. The sands were oven dried before sieving, and water of absorption was added 30 min prior to mixing. With the exception of sand gradations, the mortars were prepared in accordance with ASTM C 87-52, "Standard Method of Test for Measuring Mortar-Making Properties of Fine Aggregate."

TABLE 1—GRADING OF SANDS

Sieve size	Percent retained
No. 4	0.0
No. 8	15.0
No. 16	36.0
No. 30	58.0
No. 50	80.0
No. 100	96.0

Sample 1 was a rounded beach sand and Sample 2 was a manufactured sand. Samples 3, 4, and 5 consisted of natural river sands; their only major variation was in the specific gravity—particle shape and surface texture being similar, as indicated by examination under a microscope. Unit weights and void content of the sands were determined in accordance with ASTM C 29-55T and C 30-37, respectively.

Physical data for the five sands tested and characteristics of the fresh mortar are given in Table 2.

The data indicate that the sand-cement ratio of mortar, for a given water-cement ratio and consistency, has a linear relation to the angularity of the sand, and therefore the more rounded sands require less water for a given consistency.

Since no effort was made to differentiate between the relative effects of particle shape versus surface texture for any given mortar, it is not known to what extent each individually would affect the water requirement.

A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 5, Nov. 1958, *Proceedings* V. 55. Separate prints of the entire Concrete Briefs section are available at 35 cents each. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

*Member American Concrete Institute, Research Engineer, The Master Builders Co., Niles, Calif.

TABLE 2—CHARACTERISTICS OF SAND AND FRESH MORTARS

Sample No.	Sand				Mortar	
	Specific gravity	Fineness modulus	Unit weight, lb per cu ft	Percent voids	Flow, percent	Sand-cement ratio
1	2.58	2.85	114.7	28.8	98	4.2
2	2.53	2.85	98.0	38.0	101	2.8
3	2.59	2.85	107.5	33.4	102	3.5
4	2.65	2.85	112.3	32.1	97	3.6
5	2.54	2.85	105.6	34.4	103	3.3

It would therefore seem logical that an improvement in the shape and surface texture of the sand would result in significant reduction in the unit water content of concrete mixes. However, our experience indicates that, everything else being equal, better compressive strengths, and particularly better flexural strengths, are normally secured with sands that have some degree of sharpness as compared to rounded, smooth sands. In view of these experiences one may not conclude that the water reduction secured through the use of a round, smooth, sand will produce the most satisfactory concrete for all purposes. However, there may be some optimum condition of particle shape and surface texture that will result in optimum water reduction and optimum strength.

Concrete's Etymological Offspring*

By PETE CRETE

An ACI member with philological inclinations submits ten additions to the "concrete word family," this time including one sanctioned by Webster. The term "Rubber-Crete" is a duplication of name but apparently not of product.

Word	Where used
Con-Trow'l	(Troweling machine) Champion Manufacturing Co., St. Louis, Mo.; ACI JOURNAL, Aug. 1956, News Letter, p. 25
Faircrete	(Concrete conveyor) Fairfield Engineering Co., Marion, Ohio; *Contractors and Engineers*, May 1957, p. 158
Hydracrete	(Hydrophobic cement; formerly *Aquacrete*) Cement Marketing Co., London; *The Engineer*, Feb. 8, 1957, p. 204
Krete-Kut	(Abrasive blades) Bay State Abrasive Products Co., Westboro, Mass.; *Contractors and Engineers*, Jan. 1957, p. 100
Maracon	(Admixtures) Marathon Corp., Rothschild, Wisc.; ACI JOURNAL, Apr. 1957, News Letter, p. 45
Pectacrete	(Hydrophobic or waterproofed cement) Hydrophobic Cement Ltd., London; *Civil Engineering and Public Works Review*, Mar. 1957, pp. 330-331
Prescon	(Prestressing system) Prescon Corp., Corpus Christi, Tex., ACI JOURNAL, July 1956, News Letter, p. 21; *Engineering News-Record*, June 6, 1957, p. 86
Rubber-Crete	(Rubber-containing floor surfacing mix and/or terrazzo) Ascon Products Co., Niantic, Conn.
Silcrete	(Superficial quartzite formed by cementation of soil, sand, gravel, etc., by silica) *Webster's New International Dictionary*, 2nd edition
Tite-Crete	(Admixture) Surface Engineering Co., Inc., Wichita, Kan.; *Contractors and Engineers*, Dec. 1956, p. 63

*Previous lists appeared in ACI JOURNAL, *Proc.* V. 47, Nov. 1950, p. 263, May 1951, p. 752; *Proc.* V. 49, Mar. 1953, pp. 682-684; *Proc.* V. 50, June 1954, pp. 894-897; *Proc.* V. 51, Apr. 1955, pp. 813-817; *Proc.* V. 53, Aug. 1956 pp. 226-228; *Proc.* V. 54, Aug. 1957, pp. 166-167.

Problems and Practices

A series relating to "down-to-earth, everyday" concrete problems which attempts to give brief answers to the more common (and sometimes uncommon, too) questions asked about concrete design and construction practices.

To some, the answers will seem simple and obvious; others may prove controversial.

All ACI members are invited to participate—either by submitting an inquiry, or even better, by telling JOURNAL readers how an intriguing problem was solved. It may well be that readers will be able to suggest more practical solutions than those presented.

Q. In testing cores that have been cut from reinforced concrete buildings, what is the effect of reinforcement in the core on the test strength of the concrete? Is there any standard comparison between the strengths of a given concrete mixture, with and without reinforcement?

A. There is probably no feasible way to determine the effect upon the compressive strength of a core of reinforcement at any angle other than parallel to the axis of the core. For reinforcement in other positions the effect might well be to cause the splitting of the core, decreasing its ultimate strength; to partially restrain a typical shear failure, increasing the strength of the core; or neither, having no effect on the strength of the core. The contribution of additional compressive strength by a piece of reinforcement parallel to the core axis might be *estimated* by methods of ultimate strength design* for short tied columns.

However, the preferable procedure would be to reject any core which contained reinforcement. Whenever cores are taken from a structure several specimens should be secured, so that duplicates are available if reinforcement renders one unfit for test.

One Institute member reports: "Over a period of years the writer has noticed the effect of concrete reinforcement on the compressive strength results of cores obtained from structures.

"The results of these observations have led me to conclude that in most cases the presence of one piece of reinforcement in a horizontal or nearly horizontal position lowers the compressive strength determination and renders the test inconclusive. It does seem obvious, however, that a situation could be such that the apparent strength might be increased.

"In our laboratories we have, therefore, attempted to obtain additional samples and to reject cores containing any steel. If no duplicate samples were available we have cut off the portion containing steel and tested the remainder, even though decreasing the length of the core. Proper corrections, of course, were made. If the steel is in a vertical position no attempt has been made to test the sample."

* * *

Q. A sand cement manufactured outside the United States is under consideration for general use in place of portland cement, ASTM Type I. Test results show that the sand cement meets all chemical requirements of ASTM C 340-55T for Type IP portland-pozzolan cement and the physical requirements for fineness, setting time, soundness, air content, and water requirement. Mortar cube compressive strengths equal those for the Type I cement. The sand is principally feldspar and quartz and is used in amounts of 12 to 18 percent, but has only slight pozzolanic activity. Can this sand cement properly be called portland-pozzolan and can it be recommended for general structural use or is it particularly suited for some special use?

A. The sand cement cannot be said to have met the requirements of the ASTM specification for portland pozzolan cement because no shrinkage test was made nor was a pozzolanic activity test made on the siliceous material which was added to the cement clinker. The nature of the siliceous material is such, however, that it would appear to have little or no pozzolanic activity, as stated. The cement is in fact a sand cement not unlike those which were used in

*ACI-ASCE Committee 327, "Ultimate Strength Design," ACI JOURNAL, Jan. 1956, *Proc.* V. 52, pp. 505-525.

the construction of a number of U. S. dams very early in this century, although the percentage of sand is lower, particularly than in some of the early constructions of the U. S. Reclamation Service including Arrowrock Dam and Elephant Butte Dam. The high strength exhibited by the sand cement may be entirely accounted for by its very high fineness, the sand acting as a grinding aid.

For the sand cements used in the United States where portland cement was interground with disintegrated granite at Arrowrock and with soft sandstone at Elephant Butte, at the job site, even for a 50-50 mixture, the mortar strengths were about as high as those obtained with a straight portland cement. Concrete strengths, however, were relatively much lower.

The use of portland pozzolan cement (ASTM C 340-55T, Type IP) in the United States is not yet widespread. The establishment of the specification for this type of cement is relatively recent (tentative in 1955). Most of the building codes, architects' specifications, engineers' specifications, governmental agencies' specifications, etc., do not yet make provision for use of this cement. Several of the technical committees of the American Concrete Institute which prepare recommendations for building code requirements, pavement specifications, concrete in housing, prestressed concrete, precast concrete, winter concreting, hot weather concreting, etc., include in their recommendations a specification for cement suitable in each of these fields. As yet, none of these committees have included portland pozzolan cement Type IP. However, the most recently adopted ACI standard, "Specifications for Concrete Pavements and Concrete Bases (ACI 617-58)," does make provision for the use of portland pozzolan cement as a *blend* with portland type cement for concrete pavements.

The use of pozzolan as a separate material to be introduced at the mixer has become quite general in mass concrete construction such as dams and in other work where reactive aggregates are encountered. In some of our cities of the Middle West and East, fly ash (a pozzolan) is extensively employed in all types of construction. In small amount, diatomaceous earth (a very active pozzolan) has long been used to improve workability and inhibit bleeding.

* * *

Q. What is the detrimental influence of mica in concrete aggregates and the critical percentage if limits have been established?

A. Mica itself is not ordinarily considered deleterious. Expanded mica is a standard lightweight aggregate useful for insulating or fireproofing purposes. There is no chemical incompatibility between mica and cement. It is however a physically softer material than ordinary natural aggregates. Mica present in minor amounts in a natural hard aggregate will affect strength and probably durability under freezing and thawing.

Fine aggregates containing mica have usually been looked upon with suspicion. Very little information is available, however, on the quantitative influence of mica on the properties of concrete. Usually mortar strengths may be expected to decrease as the proportion of mica in fine aggregate increases. Limited data on this phenomenon is included in a paper in *Western Construction News*, Aug. 25, 1927, p. 47.

Minor amounts of mica occurring as relatively small inclusions disseminated throughout the particles of coarse aggregate would not usually be considered harmful. On the other hand, rock types in which mica occurs in such proportions and in such character as to cause the individual aggregate particles to be structurally weak would normally be considered unsuitable for use in concrete.

Instead of relying on critical percentage it is preferable to evaluate aggregate containing mica thoroughly by laboratory testing including tests on concrete made with a particular aggregate as a means of judging its suitability for a particular use.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Design and construction of the Hindiya Bridge

JOHN F. C. SWANSBOURNE, *Proceedings*, Institution of Civil Engineers (London), V. 8 (session 1956-57), Sept. 1957, pp. 1-16, plates

Reviewed by ARON L. MIRSKY

Bridge, comprising ten reinforced concrete spans and one centrally located steel tied-arch, crosses the Euphrates at Hindiya, 100 km south of Baghdad, in Iraq. The five approach spans at either end of arch are continuous and monolithic with the piers and the hollow-type abutments; box girders, beams, pier shells, and other elements were precast.

Bridges on German federal interregional highways. II—Massive bridges (Brückenbauten auf Bundesfernstrassen. II—Massive Brücken)

W. KLINGENBERG, *Der Bauingenieur* (Berlin), V. 32, No. 9, Sept. 1957, pp. 325-333.

Reviewed by ARON L. MIRSKY

This portion of paper covers arches, multiple-arch viaducts, frames, prestressed concrete bridges, and other interesting recent concrete structures of large size. (For first part, see *Der Bauingenieur*, July 1957, pp. 245-258; "Current Reviews," ACI JOURNAL, Oct. 1958, p. 519.)

Bridging the Strait of Gibraltar (in Dutch)

Polytechnisch Tijdschrift (Haarlem), V. 13, No. 25-26, June 1958, pp. 448-450

Reviewed by JOHN B. SNETHLAGE

This 3-page illustrated article describes a plan for a 25-span (each 1000 meters) suspension bridge designed by Alfonso Peña Boeuf, chief engineer of roads, canals and harbors of the Republic of Spain. A location plan, with underwater contours, shows the bridge site westward from the narrowest point (14 km) where the greatest water depth is said to be 1000 m. At the more westward site, bridge piers up to 400 m deep are planned and illustrations show how this can be accomplished. Don Alfonso is said to estimate the cost of the bridge at 8 billion pesetas.

Construction

Building with tilt up

F. THOMAS COLLINS, Know How Publications, Eugene, Ore., 2nd Edition, 1958, 160 pp., $10.00

Describes and illustrates proven field procedures in the construction of precast and tilt-up reinforced concrete buildings. Covers common sources of trouble and explains field procedures for avoiding trouble. Includes discussions on advanced planning, soil and foundations, floor slabs, wall panels, hot and cold weather problems, pick up and erection, bracing, and joinery. Written especially for contractors, it should be equally useful for designers to insure practicable and economical designs.

Warped deck garage construction

E. M. KHOURY, *Parking*, Spring 1958, pp. 13-14

HIGHWAY RESEARCH ABSTRACTS July 1958

The design discussed here shows a multi-story parking garage structure without the conventional ramps or spiralling ramps, usually associated with garage structures. Access to upper floors and exit from upper floors is accomplished by warping the floors

A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 5, Nov. 1958 (*Proceedings* V. 54). Address P.O. Box 4754, Redford Station, Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are **not** available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

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in such a manner as to bring this condition about. The design differs considerably from the conventional garage designs of the ramping type.

The warped deck is constructed as a hyperbolic paraboloid.

Housing problem and the industrialization of construction (Le problème du logement et l'industrialisation de la construction)

EDOUARD FOUGEA, *Mémoires*, Société des Ingénieurs Civils de France (Paris), V. 3, No. 1, Jan.-Feb. 1958, pp. 7-22

Reviewed by ARON L. MIRSKY

Postponed construction, wartime destruction, normal expansion and postwar migration, and other factors have resulted in a large demand for dwellings in France. To fill this demand, at reasonable costs, factory-type prefabrication of building elements and mechanized erection methods are necessary. Paper surveys all facets of problem and its solution. While viewpoint is French, problem is, of course, not entirely foreign to the American scene, and solution may well repay study.

Design

Plastic theory of curved beams with compressive axial forces

P. LANGE HANSEN, *Bulletin* No. 7, Research Laboratory of Building Technique, Technical University of Denmark, Copenhagen, 1957, 32 pp., 7.50 D.kr., reprinted from *Acta Polytechnica* (Copenhagen), No. 215, 1957 (Civil Engineering and Building Construction Series, V. 4, No. 1)

AUTHOR'S SUMMARY

Tests showed that in cases where the beam had compressive axial forces, the collapse load could be considerably lower than the theoretical value calculable in the original geometry of the structure. Deviations were found ranging between 1 and 21 percent.

A method is indicated transforming the difficult problem of plastic instability to a stress problem, assuming the bending moment at all the plastic hinges to be the full plastic moment. The method consists in first determining the structural deflections in the collapse stage and next determining the collapse load in the deflected structure. The deflections are determined by assuming the beam to consist of localized plastic hinges and elastic regions in between. A numerical example is given.

The results of the method have been compared with the tests published, the data of which show good agreement—within a range of 5 percent—between calculated and actual collapse loads.

On the application of the theory of bending of orthotropic plates to the analysis of skewed stringer bridges (Über die Anwendung der Biegungstheorie orthotroper Platten auf die Berechnung schiefer Balkenbrücken)

M. NARUOKA and H. YONEZAWA, *Der Bauingenieur* (Berlin), V. 32, No. 10, Oct. 1957, pp. 391-395

Reviewed by ARON L. MIRSKY

Method of finite differences was applied to analysis of skewed T-beam type bridges (without curbs), considered as orthotrophic plates. Results are in good correspondence with those obtained by analytical solution. Test of a model, in cast iron, of a bridge with a 45 deg skew gave results in good agreement with values obtained by authors' method, as distinguished from those obtained by other methods (e.g., neglecting effcte of the skew).

On Mohr's law (Über den Mohrschen Satz)

FERDINAND SCHLEICHER, *Der Bauingenieur* (Berlin), V. 32, No. 10, Oct. 1957, pp. 369-377

Reviewed by ARON L. MIRSKY

An interesting presentation of Mohr's classic analogy between the elastic curve and the funicular polygon. First published in 1868, this analogy has been the basis of much of our structural engineering. Readers familiar with the various methods in use today, as given in, say, Timoshenko and Young's *Theory of Structures* (McGraw-Hill), will find in this outline an excellent summary of the growth of Mohr's analogy, much of it due to Mohr himself.

On the limit design of reinforced-concrete plates (in Hungarian)

O. HALASZ, *Magyar Tudomanyos Akademai* (Budapest), V. 19, No. 1/3, 1956, pp. 227-237

APPLIED MECHANICS REVIEWS Aug. 1958 (J. Barta)

Aim of paper is to determine limit load. The plate is simply supported. The steel bars are placed perpendicularly on each

other. The train of thought differs from one of the fundamental theorems of the limit design theory, which says that the intensity of the limit load is intensity of that statically possible load, the bending moments of which are not greater than the limit bending moments. A detailed computation is executed for the rectangular plate.

Safety factor in eccentrically stressed ferroconcrete structural components (in Russian)

M. I. Bychkov, *Trudy Ural'skogo Politekhnicheskogo Instituta* (Sverdlovsk), No. 54, 1955, pp. 117-125

Applied Mechanics Reviews Aug. 1958 (N. S. Kurdin)

Using known formulas for calculation, author obtains an inequality $k_e/k_N \gtrless 1$ where k_e is the safety factor through the eccentricity, k_N is the safety factor through the longitudinal force, the signs of inequality $<,>$ relate to the eccentric stress of the first and second degree. Here two variants of rupture are examined: (1) the longitudinal force increases, while the eccentricity remains constant; (2) the longitudinal force does not change, while the eccentricity increases. Examples are put forward, and a short survey is given of the works connected with the coefficient of safety for eccentrically stressed ferroconcrete structural members.

Materials

Frost resistance of cement paste as influenced by surface-active agents

Ulf Danielsson and Anders Wastesson, *Proceedings* No. 30, Swedish Cement and Concrete Research Institute (Stockholm), 1958, 38 pp., 5 kr.

Authors' Summary

The effect of five different surface-active agents on the frost resistance of cement pastes of varying water-cement ratios has been investigated. The results indicate that the frost resistance is mainly determined by the volume and size distribution of the air-filled pores in the pastes, while chemical effects are of minor importance.

A theory—the theory of the frost-protected volume ratio—is advanced to explain frost damage, based upon the same fundamental assumptions as the hydraulic pressure theory put forward by T. C. Powers. The theory takes the actual size-distribution of the pores into account, instead of assuming them to be of the same size. The frost resistance is expressed in terms of the frost-protected volume per unit volume of paste, v_{pr}, which is approximately:

$$v_{pr} = A\,(1 + K\alpha)$$

where A is the total air content per unit volume of paste, α the mean specific surface area of the pores, and K is a coefficient of dimension length which depends on the water-cement ratio and age of the paste, the freezing conditions, and other factors. This formula represents the two first terms of a series and has been found to give a fair estimate of frost resistance.

It follows that admixtures which give small and well separated pores are preferable, but because of their effect on the workability, other admixtures may be of comparable merit, due to the possibility they offer of decreasing the water-cement ratio, and thus improving the frost resistance.

Chemical reactions of aggregate in concrete

Special Report No. 31, Highway Research Board (Committee on Durability of Concrete—Chemical Aspects), 1958, 12 pp.

Identifies minerals or rocks known to cause deleterious chemical reactions in concrete. Lists the geographical location of known reactive aggregates in the United States; describes various test methods to identify them and discusses recommended procedures for their use. Discussions include alkali-aggregate reaction and cement-aggregate reaction.

Setting, strength, and aeration of supersulfated cement

Katsuhiko Asaoka and Eizo Sano, *Semento Gijutsu Nenpo*, V. 9, 1955, pp. 139-145

Ceramic Abstracts Aug. 1958 (Y. Suzukawa)

The cements were prepared by mixing slag 80 to 85 percent, various forms of gypsum 15 percent, and clinker 0 to 5 percent by weight. Their physical properties were compared with those of portland and portland blast furnace slag cements. The cement consisting of 83 percent slag, 15 percent gypsum, and 2 percent clinker by weight showed superior strength. Among various forms of gypsum, synthetic anhydrite calcined at 850 C is most desirable next to natural anhydrite. When these cements

were aerated for 3 months, the mortar strength of cements containing 2 percent by weight of portland cement decreased considerably, but that of cements containing 5 percent by weight increased about two to five times that of the original cements.

False set of portland cement—chemical composition of the liquid extracted from neat cement paste

Yoshikazu Murakami, *Semento Gijutsu Nenpo*, V. 10, 1956, pp. 61-67

Ceramic Abstracts
Sept. 1958 (Y. Suzukawa)

False set due to $CaSO_4 \cdot 1/2H_2O$ can be explained by determining the chemical composition of the liquid extracted from neat paste of normal consistency immediately and at 10 min after mixing. No explanation was found, however, for the false set of portland cement containing $CaSO_4 \cdot 2H_2O$.

Effect of blast furnace slag on the water permeability of mortar

Naoshi Kubo and Takeo Suzuki, *Semento Gijutsu Nenpo*, V. 11, 1957, pp. 156-162

Ceramic Abstracts
Sept. 1958 (Y. Suzukawa)

Water permeability of mortar decreases when the cement is replaced by slag of superior quality, and it shows a minimum value at 30 to 50 percent by weight replacement. Fly ash is inferior to slag for decreasing water permeability.

Effect of particle-size distribution of cements ground in a test ball mill on their mortar strength

Shigejiro Mori and Toshio Kawano, *Semento Gijutsu Nenpo*, V. 10, 1956, pp. 156-164

Ceramic Abstracts
Sept. 1958 (Y. Suzukawa)

Determination of particle-size distribution by the air elutriation method showed that, in the scope of the present experiments, the value of n in the exponential formula ($R = e^{-kx^n}$) is affected principally by the amount of particles $< 15\mu$ and, in cements having the same value of k, the specific surface (Blaine) increases with increasing value of n. The effects of the amount of particles $< 15\mu$, 15 to 40μ and $> 40\mu$ on mortar strength development were determined by the method of least squares. It was found that the particles $< 15\mu$ contribute the strength at earlier ages, especially at 3 days, and the particles of 15 to 40μ are effective for strength development at later ages. Compressive strength of cements having the same specific surface and a different value of n showed no differences at 1 day, but the higher the value of n, the greater was the strength and heat of hydration at later ages.

Effect of the fineness of slag and clinker on the properties of portland blast furnace slag cement

Chokui Saito and Toshitsugu Iwabuchi, *Semento Gijutsu Nenpo*, V. 11, 1957, pp. 150-156

Ceramic Abstracts
Sept. 1958 (Y. Suzukawa)

When slag or clinker was charged in a test mill (480 x 620 mm) so that the same volume percentage of the voids in the grinding media was occupied by them, no difference in their grindability was observed during coarse grinding, but the fine grinding of slag was easier than that of clinker. Drying shrinkage of the cement mortar increased with increasing fineness of slag and clinker, the effect of the fineness of the clinker being especially greater. No difference in the resistance of concrete specimens to rapid freezing and thawing in water was observed between the original portland cement and the portland blast-furnace slag cements containing up to 50 percent by weight of slag.

Effectiveness of mineral admixtures in preventing excessive expansion of concrete due to alkali-aggregate reaction

Alan D. Buck, B. J. Houston, and Leonard Pepper, *Technical Report* No. 6-481, Waterways Experiment Station, Vicksburg, Miss., July 1958, 54 pp., $0.75

Authors' Summary

Twenty materials, representing eight different classes of mineral admixtures, were evaluated, using both chemical and mortar bar test methods, for their effectiveness in preventing excessive expansion of concrete due to alkali-aggregate reaction. It was found that the chemical tests cannot be used with reliance to evaluate effectiveness, and the mortar bar test method needs improvement to increase its precision. Each of the replacement materials evaluated will prevent excessive expansion if a sufficient quantity is used. Correlations were found between

effectiveness and (1) fineness, (2) dissolved silica, and (3) percentage of alkali retained by reaction product. The eight classes of mineral admixtures represented in the tests were: granulated blast furnace slags, natural cements, fly ashes, natural volcanic glasses, calcined opaline shales, uncalcined diatomite, uncalcined quartz, and synthetic pure silica glass.

Characteristics of false set not due to the setting of calcium sulfate hemihydrate

Yoshikazu Murakami, *Semento Gijutsu Nenpo*, V. 11, 1957, pp. 61-66

Ceramic Abstracts
Sept. 1958 (Y. Suzukawa)

False set of portland cement cannot be explained only by the setting of $CaSO_4 \cdot 1/2H_2O$, because cement containing $CaSO_4 \cdot 2H_2O$ also shows false set. False set of the latter type is observed when the cement is aerated slightly in air of lower humidity, but the cement again shows normal set when placed in air of higher humidity. False set is very sensitive to the testing temperature, i.e., the cement changes from false set to normal set when the testing temperature is lowered to 2 to 3 C.

Effect of initial curing temperature of neat cement paste on the degree of hydration

Shunsuke Takano, Kenichi Oishi, and Osaku Matsuda, *Semento Gijutsu Nenpo*, No. 9, 1955, pp. 78-84

Ceramic Abstracts
July 1958 (Suzukawa)

Determinations of compressive strength, porosity, and $CA(OH)_2$ content showed that the lower the initial curing temperature, the denser is the structure of the hardened paste, the higher is the strength, and the greater is the degree of hydration.

Investigations of Danish aggregates at Building Research Station of England

F. E. Jones, *Progress Report* I 1, Committee on Alkali Reactions in Concrete, The Danish National Institute of Building Research and the Academy of Technical Sciences, Copenhagen, 1958, 62 pp., 12 D.kr. (25 percent discount may be obtained by ordering full series)

Reports on laboratory and field investigation of alkali-aggregate reactions. Most of the distressed concrete examined in the field study was located on various Danish bridges. Laboratory investigation divided reactive aggregate into two classes, that which would cause harmful expansion, and that suitable for use provided no additional source of alkali other than cement would be present in service. It was found that some of the bridges constructed with the latter type of aggregates deteriorated only when exposed to sea water.

Pozzolith (in Dutch)

J. van Zutphen, *Polytechnisch Tijdschrift* (Haarlem), V. 13, No. 27-28, June 1958, pp. 447-484
Reviewed by John B. Snethlage

Article with photographs, tabulations, curves, and graphs, describes the effect of pozzolith on concrete, particularly with respect to workability, strength, bond to steel, shrinkage, watertightness and durability. Pozzolith is manufactured under license by Pozzobel S.A. in Antwerp.

Statistical study of flexural strength of concrete beams

William C. Otto, *Proceedings*, ASTM, V. 57, 1957, pp. 1154-1167; discussion pp. 1168-1169

The variability of flexural strength of concrete for airfield pavement is considered. All of the data relate to eight airfield projects. The concrete pavements represented by the data include runways, taxiways, parking aprons, and warm-up aprons.

Slip-form paver lays reinforced slab

George N. Miles, *Roads and Streets*, V. 101, No. 8, Aug. 1958, pp. 53-55

A mile of experimental concrete pavement was placed in Colorado demonstrating that reinforcement need not deter the use of slip-form pavers. The contractor developed a special "sled" device that was attached to the front of the slip-form paver to handle the wire mesh. Initial difficulties with the method and their solution are described.

Review of available information on polyvinyl acetate as an admixture for concrete

Clara F. Derrington and Leonard Pepper, *Technical Report* No. 6-486, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., July 1958, 11 pp., $0.50

Author's Summary

Presents a review of available literature pertaining to the incorporation of polyvinyl acetate as a concrete admixture into mortar

or concrete mixtures. Polyvinyl acetate has been reported to produce excellent bonding of the fresh mortar to old concrete and to improve several properties of concrete. This review of available literature suggests that mortar containing PVA exhibits improved physical properties when the modified concrete is cured in a dry atmosphere. However, when mortar containing PVA is immersed in water or subjected to weather conditions involving high humidities, considerable loss of strength, warping, and cracking result.

Lignin as a retarder for concrete

M. T. Loughborough, *Ontario Hydro Research News* (Toronto), V. 10, No. 1, Jan.-Mar. 1958, pp. 15-19

Author describes conditions of weather or special placement requirements where retardation of the rate of stiffening of concrete is desirable. The effects of lignin to accomplish this result as well as its effect on water requirement, air entrainment, and compressive strengths of the concrete are included.

Conclusion is that lignin can be useful for controlling the rate of stiffening of concrete and that adjustment of the dosage gives a wide range in degree of retardation. Concrete control personnel must assume additional responsibilities to insure proper use of admixtures.

Pavements

Design of continuously reinforced concrete pavement

Wayne R. Woolley, *Bulletin* No. 181, Highway Research Board, May 1958, pp. 1-4

Compares results by presently available design theory to field performance. Concludes that design theory is inadequate but that performance justifies a minimum of ½ percent reinforcement.

Highway division experiments with slab reinforcing

John L. Beaton, *Southwest Builder and Contractor*, V. 131, No. 15, Apr. 1958, pp. 47-48, 50, 51, 60
Highway Research Abstracts July 1958

Result of a study on reinforcing steel by the California Division of Highways was to recommend: (1) For fabricating caps, or other non-stress-carrying members, any type of welding that did not result in undercutting, cracking, or hardening of weld area could be used. (2) For major stress-carrying reinforcing steel, either pressure gas, flash, or manual welding could be used. Because of economy, it is considered that manual welding would be the most practical. Manual welding should use low hydrogen electrodes (E7015 and E7016 for intermediate grade and E8015 or E8016 for hard grade). Bars of the #14 or #18 size should be preheated to 400 F in the weld area.

Preliminary report on continuously reinforced concrete pavement research in Pennsylvania

R. L. Schiffman, I. J. Taylor, and W. J. Eney, *Bulletin* No. 181, Highway Research Board, May 1958, pp. 5-20

A preliminary report on construction procedures and test measurement details on two continuously reinforced concrete pavements. Studies are underway on crack frequency and crack width related to slab temperature. The paper describes results obtained during the first 2 months of the pavement's life.

Energy-absorbing barrier for highways

Norman L. Pedersen, John H. Mathewson and Drewyn M. Severy, *Bulletin* No. 185, Highway Research Board, May 1958, pp. 19-29

Describes a reinforced concrete energy-absorbing barrier system consisting of a series of corrugated concrete slabs designed to absorb approximately 2000 ft-lb at usual impact speeds and reinforced to prevent severe fragmentation.

Precast Concrete

Measuring shrinkage of concrete block —a comparison of test methods

E. L. Saxer and H. T. Toennies, *Proceedings*, ASTM, V. 57, 1957, pp. 988-1005; discussion pp. 1006-1011
Authors' Summary

Concrete masonry units, made with five different aggregates and cured by low pressure steam or by autoclaving, were tested for drying shrinkage by three different methods. Auxiliary tests were made on prisms cut from companion block. Physical properties of the block and the aggregate were determined.

The data indicate that the "modified British test" on whole block units could be

substituted as an accelerated test method of measuring concrete masonry shrinkage as it occurs during normal atmospheric drying.

Shrinkage measurements by the accelerated "rapid test" were substantially greater than those occurring with normal drying. No single correlation, applicable to all block, could be established between "rapid test" measurements and normal drying shrinkage, but a fair correlation was noted for block manufactured by a given curing technique.

Shrinkage measurements on cut prism sections were inconsistent and showed poor correlation with results obtained on whole block.

Industrial production of precast concrete structural units in Denmark

ARNO JENSEN and JANNIK IPSEN, *Ingeniøren*, C Edition (Copenhagen), V. 2, No. 2, May 1958, pp. 53-65

Reviewed by JESPER STRANDGAARD

The illustrated article gives a brief account of the production of precast concrete units, conventionally reinforced and pretensioned. The phases of production, design, planning, form making, concrete mixes, vibration, curing, and surface treatment are described, and a variety of examples of uses of precast units are given. The standardization of certain types of beams is shown together with tables of load capacity, and this standardization is credited with the wide use of such beams by the building industry in Denmark.

Prestressed Concrete

Prestressed concrete in Italy (El hormigón pretensado en Italia)

FRANCO LEVI, *Bulletin* No. 184, Instituto Técnico de la Construcción y del Cemento, Madrid, 1957, 26 pp.

Reviewed by ROGER DIAZ DE COSSIO

A general picture of the status of prestressed concrete construction in Italy today. Mention is made of different applications of prestressing techniques including shell roofs, frames, bridges, and warehouses. Of special interest is the description of the erection of a 235-ft three-hinged reinforced concrete arch. The two halves of the arch were cast at the sides of the river and then rotated around the supports with the help of temporary prestressed cables of variable tension to diminish the stresses caused by the movement. The author makes some comparisons between "static" and continuous constructions, and discusses the uses of temporary "hinges" while the members are erected. Economic considerations on reinforced and prestressed concrete are also given.

Prestressed concrete—theory and design

R. H. EVANS and E. W. BENNETT, John Wiley & Sons, Inc., New York, 1958, 294 pp., $10

Part 1 (covering prestressing and flexure under working load, prestressing force, flexural strength, and shear and principal tensile stresses) is devoted to the basic principles which apply to all types of prestressed concrete structures. Part 2 deals with the design of simply supported beams. The aim of this section is to bridge the gap between principles and practice and the maximum possible use is made of formulas, tables, and graphs. Part 3 introduces specialized types of prestressed structures and outlines the main principles. Special applications covered are: composite construction of prestressed and cast-in-place concrete, miscellaneous statically determinate structures, indeterminate structures, liquid-retaining structures, and domes and shells.

The notation in the book is based on the recommendations of the First Report of the Institution of Structural Engineers of England. However, the authors have included in their table of notation the equivalent American symbol for each item where one exists.

Properties of Concrete

Studies of flexural strength of concrete—Part 3: Effects of variations in testing procedures

STANTON WALKER and DELMAR L. BLOEM, *Proceedings*, ASTM, V. 57, 1957, pp. 1122-1139; discussion pp. 1140-1142

AUTHORS' SUMMARY

The indicated flexural strength of concrete is more significantly affected by relatively obscure factors than is commonly realized. The investigation described provides data on modulus of rupture for: different positions of beams as molded; five methods of applying load; beams of different cross sections; sawed versus molded beams; and different moisture conditions of specimens and different

ages at test. Data are also presented on reproducibility of duplicate tests on the same beam and comparisons between flexural and compressive strength. The data point to the need for a review of current testing procedures. They suggest also that, while the flexural strength test is a useful tool in research and in the laboratory evaluation of concrete ingredients and proportions, it is too sensitive to testing variations to be usable as a basis for the acceptance or rejection of concrete in the field.

Part 1 of this study was reviewed in the October 1956 "Current Reviews" section, p. 431.

Effect of calcium chloride on sulfate resistance of mortar and concrete

NOBUE FUKUDA and IWAO YAMADA, *Semento Gijutsu Nenpo*, V. 10, pp. 182-188

CERAMIC ABSTRACTS Sept. 1958 (Y. Suzukawa)

A small increase or decrease in the sulfate resistance of mortar cured in 10 percent solution of Na_2SO_4 or $MgSO_4$, respectively, was observed when 2 percent by weight of anhydrous $CaCl_2$ was added to a cement. The $CaCl_2$ seems to have little influence on the sulfate resistance of concrete up to the age of 6 months.

Instrumentation for measuring characteristics of concrete

Bulletin 176, Highway Research Board, Apr. 1958, 28 pp., $0.60

This bulletin contains three papers presented at the 36th Annual Meeting of the Highway Research Board.

The first, by G. E. Monfore, describes a new type of standardizing unbonded elastic wire strain gage that can be surface-mounted or embedded in concrete for measurements requiring long-term stability.

The second, by George Verbeck and Paul Klieger, describes the development and operation of a calorimeter-strain apparatus for study of freezing and thawing concrete, especially the determination of (a) the amount of water actually freezing within a concrete specimen as a function of temperature and time, and (b) the physical response of the specimen in terms of length change as this water is frozen.

The third, by W. E. Grieb, describing the AE-55 indicator is reviewed next.

AE-55 indicator for air in concrete

W. E. GRIEB, *Bulletin* No. 176, Highway Research Board, Apr. 1958, pp. 23-28

Presents comparisons on the measurement of entrained air in concrete by standard methods and by the "Chace" pocket meter. Corrections were determined for the pocket air indicator and with these corrections it was concluded that it would be very useful as a supplementary test but that it should not be considered suitable for replacing standard methods. Includes a discussion presenting further data in agreement with the original.

Long-term expansion of perlite plaster and concrete

R. D. HILL, *Australian Journal of Applied Science* (Melbourne), V. 9, No. 2, 1958, pp. 141-162

AUTHOR'S SUMMARY

A study has been made of five perlite aggregates and their raw materials, and of perlite plaster and concrete made with these aggregates. Evidence is presented to show that the principal cause of the long-term expansion in perlite plaster and concrete is the expansion of the aggregate upon rehydration, and that the ion exchange is, at the most, of minor importance. Factors which influence the rate of the expansion are the nature of the raw material, the density of the aggregate and its content of bound water at the time of use, the relative humidity of the atmosphere, and the hygroscopicity of the plaster. The temperature at which the raw material was processed also has an effect, probably indirect, on the rate. The causes of high rates of expansion and the means for their prevention are discussed.

Thermal conductivity of refractory insulating concrete

W. C. HANSEN and A. F. LIVOVICH, *American Ceramic Society Bulletin*, V. 37, No. 7, July 1958, pp. 322-328

AUTHORS' SUMMARY

Thermal conductivity data are given for 20 insulating and three dense refractory Lumnite cement concretes. Data for 15 concretes were found in good agreement with a curve established by the authors to show the relationship between thermal conductivity and unit weight of the concrete, whereas the data for the remaining eight concretes de-

viated from the curve by more than 5 percent, i.e., in excess of limits of accuracy of the test. The 23 concretes were selected for the tests on the basis of the desired unit weights, which were achieved by various means such as the blending of different aggregates, varying size and type of aggregate, varying the method of placing, and by using air-entraining agents.

Structural Research

Photoelastic investigations with models of reinforced concrete and prestressed concrete structures (Spannungsoptische Versuche mit Stahl- und Spannbetonmodellen)

Max Kufner, *VDI Zeitschrift* (Düsseldorf), V. 100, No. 9, Mar. 21, 1958, pp. 361-364
Reviewed by Aron L. Mirsky

An illustrated abstract of the pioneer and three recent papers: A. H. Beyer and A. G. Solakian, *Proceedings*, ASCE, V. 99, 1934, pp. 1196-1212; and two papers by H. H. Racké, *Schweizer Archiv fur angewandte Wissenschaft und Technik* (Solothurn), V. 22, 1956, pp. 150-163 and 169-177, and a dissertation by A. C. de Vasconcelos, Munich, 1955, on photoelastic investigations using plastic models with metallic inserts. No mention is made of the work of R. G. Boiten (*Applied Scientific Research* (The Hague), V. 5, No. 5, Section A, 1955, pp. 359-373; "Current Reviews," ACI Journal, May 1956, p. 1024).

Effect of surface characteristics and plastic properties of reinforcement on the redistribution of stresses in statically indeterminate reinforced concrete construction (in Russian)

S. M. Krylov and S. I. Ikramov, *Beton i Zhelezobeton* (Moscow), May 1958, pp. 183-86
Reviewed by D. Watstein

This experimental study is concerned with the behavior and mode of failure of beams continuous over two spans, reinforced with three different types of steel bars: hot rolled deformed bars, cold worked plain bars, and cold worked deformed bars. It was found that the character and degree of redistribution of stresses depend upon the surface characteristic of the bar as well as their plastic properties. The plain cold worked bar permitted the redistribution of stresses without either producing a brittle failure or without developing a strain approaching the value causing a tensile failure.

However, the cold worked deformed bar having the same plastic properties was fractured during the test of the beam. The fracture was attributed to the fact that the lugs of the deformed bar and the resulting good bond with the concrete interfered with the redistribution of stresses forcing the large strains associated with redistribution of stress to be confined to a relatively short length of the bar. This resulted in a rapid loss of the plastic properties and fracture of the reinforcement in that section.

Plain reinforcing bars which lost their bond over a considerable length as the load approached the maximum value still had considerable reserve of plastic properties at the time of destruction of the compressive zone and were able to develop redistribution of stresses.

Hot rolled deformed bars were found capable of permitting considerable redistribution of stresses without fracturing. However, it is pointed out that such reinforcement must have considerable plastic properties, otherwise its behavior will be analogous to the behavior of cold worked deformed bars.

The bending moment over the middle support was reduced about 30 percent by virtue of the redistribution of stresses, as compared with the value computed by elastic analysis, while the maximum width of cracks observed over the middle support did not exceed 0.2 mm.

General

Fiftieth anniversary—German Committee for Reinforced Concrete, 1907-1957 (in German)

Wilhelm Ernst and Son, Berlin, 1957, 124 pp., 24 DM
Reviewed by Fritz Kramrisch

To celebrate its 50th anniversary the Deutscher Ausschuss für Stahlbeton published a brochure containing the history of the organization and biographies of some of its most famous deceased members; also included is a list and digest of the test programs and reports the society has performed and published during its existence as well as a list of its committees and members.

It offers a review of the development of reinforced concrete and of the building code

requirements regulating its design and construction for which this society deserves great credit and has gained undisputed respect all over the world. A short preview of the developments that can reasonably be expected and foreseen for the future concludes this impressive brochure.

Performance tests of concrete truck mixers

ALBERT G. TIMMS, *Proceedings*, ASTM, V. 57, pp. 1012-1026; discussion 1027-1028

Concrete truck mixers of the inclined axis type were tested under field conditions. The effects of nine different batching sequences on the uniformity of both air-entrained and non-air-entrained concrete were investigated. The number of revolutions of the mixer drum was varied in steps from 50 to 150.

The results of the tests clearly demonstrated that uniform pavement concrete with slumps below 2 in. can be made and discharged reasonably rapidly without significant segregation of the ingredients. For satisfactory uniformity, the minimum revolutions of the drum recommended by the ASTM specifications appeared sufficient for the majority of the loading methods; improvement in uniformity was obtained by increasing the number of revolutions of the mixer drum. The tests indicated the importance of having water in the drum for both ribbon and split loading.

Corrosion of reinforcing steel and repair of concrete in a marine environment

Bulletin No. 182, Highway Research Board, May 1958, 41 pp., $0.80

This bulletin comprises the two parts of a paper entitled "Causes and Repair of Deterioration to a California Bridge Due to Corrosion of Reinforcing Steel in a Marine Environment."

Part I: Method of repair

M. W. GEWERTZ, pp. 1-17

Discusses the history, character, and extent of the deterioration encountered in the San Mateo-Hayward Bridge; the inspection and estimating procedures prior to repair; repair procedures; the basis of contract payment; and the costs of repairs.

Part II: Fundamental factors causing corrosion

BAILEY TREMPER, JOHN L. BEATON, and R. F. STRATFULL, pp. 18-41

Describes results of tests and observations to establish the cause of corroding reinforcement in the San Mateo-Hayward Bridge. Permeable concrete exposed to salt water spray resulted in severe corrosion of the steel causing spalling of the concrete. Some evidence of sulfate attack in the concrete is recorded. The corrosion was identified as galvanic corrosion. Two experimental methods of eliminating or retarding future corrosion under test include (a) cathodic protection and (b) application of a coating impervious to oxygen to the surface of cathodic areas of the deck units.

Appreciation of the Schmidt rebound hammer

J. KOLEK, *Magazine of Concrete Research* (London), V. 10, No. 28, Mar. 1958, pp. 27-36

AUTHOR'S SUMMARY

The Schmidt rebound hammer is described and some theoretical aspects underlying the functioning of the hammer are discussed and supported by some experimental results. The method of performing the test and its limitations under different conditions are described. Correlations with rate of wear of concrete and compressive strength on the basis of 500 cube results are derived. The Schmidt rebound hammer is recommended for use in two categories of application—on the building site and in precast works.

Investigation of the quality of thick concrete by ultrasonic pulse propagation

A. E. CAWKELL, *Magazine of Concrete Research* (London), V. 10, No. 28, Mar. 1958, pp. 23-26

AUTHOR'S SUMMARY

The velocity of ultrasonic pulses through concrete is measured by an oscillograph timing technique. The relation between velocity and concrete quality has been described elsewhere. To increase the range—about 8 ft through good quality concrete with the standard instrument (type UCT)—higher power, and low frequency sharply directional magnetostriction transducers were used.

Practical details of the tests are described, and it is considered that 20 ft is the maximum path length possible with this method.

ACI News Letter

Vol. 30, No. 5 JOURNAL of the AMERICAN CONCRETE INSTITUTE November 1958

Illinois Toll Highway Bridge . . . p. 9

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News Letter Contents

November 1958

On the Cover—Seven miles of precast, prestressed girders will be used in this twin 4900-ft bridge which carries the Illinois Toll Highway across two canals and two railroad right-of-ways. Positioning time for the prestressed girders averaged 15 min each. More details and another picture appear on p. 9 of this News Letter.

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Program plans announced

for February 23-26, 1959

THE LOS ANGELES LOCAL COMMITTEE for the '59 ACI Convention flourishes an "all clear" signal as plans for the 55th annual meeting to be held at the Statler Hilton Hotel rapidly take shape.

February 23 and 24 will be devoted to technical committee meetings, with the first general session on ACI standards and committee reports slated for Tuesday afternoon, February 24. Possible revision of Institute Bylaws will also be considered.

A full technical program will begin on Wednesday, February 25, with two sessions running concurrently through Thursday noon, to be followed by the final research session on Thursday afternoon. California authors and speakers have been given a prominent part in the convention program.

Topics to be featured at the construction session include erection of precast concrete; joinery of precast elements; discussion of concrete pavements on cement-treated subgrades in California; qualification plan for ready-mixed concrete plants; and cast-in-place concrete pipe.

At the Wednesday session on special problems in structural concrete, papers will be presented on the properties of shielding concrete, the blast resistance of concrete structures; and the influence of grade of steel on blast resistance of reinforced concrete beams. Surface precooling to prevent cracking of mass concrete is also scheduled for discussion at this session.

On Wednesday afternoon, February 25, the concurrent sessions will be devoted to design and analysis and concreting materials and methods. At the design and analysis session speakers will discuss prestressed concrete lift slab; effect of longitudinal forces on a portal frame

supporting a high bridge deck; prestressed concrete shells for grandstands and roofs; and pavements continuously reinforced with deformed bars.

Subjects such as the evaluation of concrete and mortar mixes, shotcrete, tremie concrete, and concrete and concrete materials for Glen Canyon Dam will be covered at the materials and methods session.

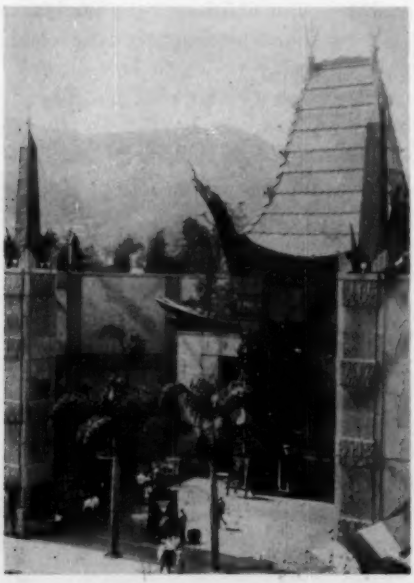
Concurrent sessions feature design research, precast units

On Thursday, February 26, design research and products and precast elements sessions will run concurrently during the morning, featuring papers dealing with the effect of bar cutoff on bond and shear strength of reinforced concrete beams; shear strength of two-span continuous reinforced concrete beams; torsional resistance of reinforced concrete; and behavior of a continuous concrete slab prestressed in two directions. "Composite Design of Pretensioned and Reinforced Concrete for Decks, Arches, and Folded Plates," is one of the papers assigned to these sessions. Decorative panels, exposed aggregate panels, glazed face concrete block, and lining for concrete pipe and structures under sewage conditions are other topics to be reported upon.

The annual research session Thursday afternoon will bring latest reports of tests on concrete in progress at institutions throughout the nation.

The Institute's honorary medals and awards will be presented at the luncheon February 25, and newly-elected honorary members of the Institute will be introduced.

C. Taylor Test, chairman of the entertainment committee, has announced plans for a Wednesday evening social hour, a trip to Disneyland on Friday, February 27, and an array of social events for the ladies including get-acquainted brunch, luncheon, and special program at the Beverly Hilton Hotel on Thursday, along with the other numerous attractions that only Los Angeles can offer.



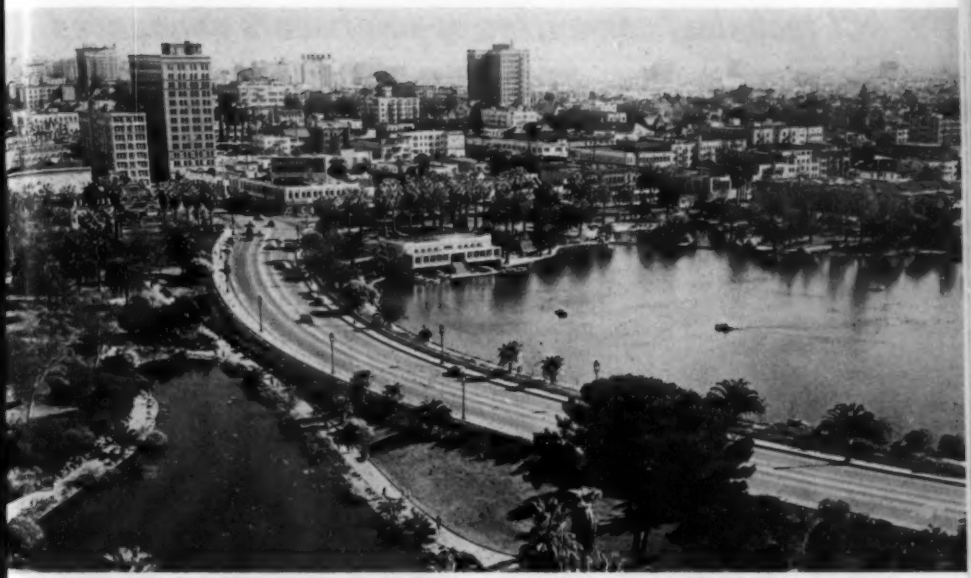
ENTRANCE TO GRAUMAN'S CHINESE THEATRE located on world-famous Hollywood Boulevard, another of the sights to be seen by ACI members attending the 1959 Los Angeles convention. This is the scene of many an exciting premiere night with hundreds of footprints and handprints imbedded in the concrete

Local committee sponsors exhibit and student competition

An exhibit will be held in conjunction with the convention, and as an added feature of the convention, the exhibits committee will hold a student competition with awards for models and other presentations depicting concrete designs and applications. Glen C. Thomas heads the exhibits committee.

Sam Hobbs, secretary-treasurer, ACI Southern California Chapter, is general chairman for the local committee. General committee correlating all planning includes John McNerney, Portland Cement Association; Byron P. Weintz, Consolidated Rock Products; William F. Norton, Ceco Steel Products Corp.; C. Taylor Test, Riverside Cement Co.; Glen C. Thomas, Thomas Concrete Accessory Co.; Lewis K. Osborn of Kistner, Wright and Wright; and Ross Adams of the Portland Cement Association.

Convention-goers who have time to include a little sight-seeing in their itinerary will find the Los Angeles area truly a tourists' mecca.



WORLD FAMOUS WILSHIRE BOULEVARD looking east across MacArthur Park in Los Angeles. Known as the boulevard of beauty, Wilshire wends its glamorous way from the center of the city westward to the Pacific. This famous "Fifth Avenue of the West" is lined with fashionable stores, churches, hotels and modern office buildings —one small part of the vast array of Southern California attractions for ACI members attending the Institute's 55th annual convention in Los Angeles, February 23-26

One can take a trip to the mountains, a boat ride to Catalina, or even a jaunt to the desert for winter sun bathing and still be in Los Angeles County.

Once famous for sunshine and scenery exclusively, Los Angeles has outgrown some of its youthful oddities and has earned a substantial place in the nation's industrial and cultural life. The vivid and new vie in sharp but beautiful contrast to the somnolent charm of Old Spain. Hollywood and Vine, world-famed intersection in the City of the Stars, is the focal point of the vast Los Angeles entertainment industry. TV and radio stations, recording studios, and movie lots are nearby.

The original heart of Los Angeles was the Plaza, laid out as a Mexican pueblo in 1781. Here in Olvera Street you can step back 200 years into history to recapture the quaint charm of Old Mexico. Almost as colorful is New Chinatown a few blocks to the southeast. Of modern oriental architecture, with landscaped courts and narrow arcades, it grew up when old Chinatown was razed to make room for the giant Union passenger terminal east of the Plaza.

Los Angeles offers cultural attractions in its Griffith Planetarium, Hollywood Bowl, and state and county museums. There are world-renowned night spots and the famous Wilshire Boulevard lined with fashionable stores, churches, hotels, and modern office buildings.

The committee promises you'll have the sun by day and the stars at night with a temperature of about 65.

Plan now to attend the 55th Annual ACI Convention scheduled for February 23-26 with headquarters at the Statler Hilton Hotel in one of America's loveliest cities, Los Angeles.

ACI technical committee appointments announced

Listed below are committee members who have recently accepted appointments to ACI technical committees. Included are *new* appointments only.

Committee 201, Durability of Concrete in Service

Inge Lyse
Norges Tekniske Hogskole
Trondheim, Norway

Committee 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concrete

Adrian Pauw
University of Missouri
Columbia, Mo.

Committee 324, Precast Reinforced Concrete, Thin Sections

David Watstein
National Bureau of Standards
Washington, D. C.

Committee 325, Structrual Design of Concrete Pavements for Highways and Airports

Gordon K. Ray
Portland Cement Association
Chicago, Ill.

Paul F. Rice
Concreting Reinforcing Steel Institute
Chicago, Ill.

Committee 326, Shear and Diagonal Tension (Joint ACI-ASCE)

William J. Krefeld
Columbia University
New York, N. Y.

Committee 331, Structures of Concrete Masonry Units

Edward Mangotich
Arizona Precast Concrete Co.
Phoenix, Ariz.

Lee Sphar
Concrete Products Association of State of Washington
Seattle, Wash.

Committee 333, Composite Construction (Joint ACI-ASCE)

James Michalos
New York University
New York, N. Y.

Committee 334, Concrete Shell Structures

Frank Baron
University of California
Berkeley, Calif.

Committee 714, Concrete Bins and Silos

Frank A. Reed
Rust Engineering Co.
Pittsburgh, Pa.

Committee 716, High Pressure Steam Curing

C. E. Lovewell
Chicago Fly Ash Co.
Chicago, Ill.

Committee 717, Low Pressure Steam Curing

C. E. Lovewell
Chicago Fly Ash Co.
Chicago, Ill.

George A. Mansfield
Moors Concrete Products
Fraser, Mich.

John A. Ruhling
American Concrete Pipe Association
Washington, D. C.

Myers Van Buren
Raymond International Inc.
New York, N. Y.

Henry Toennies
National Concrete Masonry Association
Chicago, Ill.

W. C. Green
Texas Industries, Inc.
Fort Worth, Tex.

D. K. Woodin
East Bay Municipal Utility District
Oakland, Calif.

Positions and Projects

Tobin addresses ACI Southern California Chapter

New officers of the ACI Southern California Chapter presided at a meeting at the Rodger Young Auditorium, Los Angeles, on September 17.

President Ernst Maag reported on a recent meeting of the chapter's board of direction and announced the appointment of the following committee chairmen: Finance, Hugh Barnes, Portland Cement Association; Membership, O. C. Struthers, Raymond International Inc.; Program, Dewain R. Butler, Integrated Constructors and Engineers, Inc.; Technical Activities, J. L. Peterson, J. L. Peterson, Inc.; and Convention, Henry M. Layne, structural engineer, and William T. Wright, of Kistner, Wright and Wright, as co-chairmen.

Sam Hobbs, secretary-treasurer of the ACI Southern California Chapter, left; Ernst Maag, newly elected chapter president, right, share the speaker's rostrum

Robert E. Tobin, Portland Cement Association, was speaker of the evening, basing his discussion of concrete frames and floors on design and construction methods.

Wright joins Waterloo faculty

D. T. Wright, formerly a faculty member at Queen's University, Kingston, Ont., has joined the staff at Waterloo College, Waterloo, Ont., as professor of civil engineering. Professor Wright is a member of ACI-ASCE Committee 328, Limit Design.

Ceremony marks start of official AASHO Road Test

Scientific, engineering, military, and political fields were represented at ceremonies on October 15 marking the start of traffic at the AASHO Road Test, the huge highway research project, northwest of Ottawa, Ill.

Governor William G. Stratton of Illinois opened the program with a welcome to official guests from all over the nation. Responses to the governor's welcome were made by C. R. McMillan, president of the American Association of State Highway Officials, sponsor of the project; and Charles H. Scholer, chairman of the Highway Research Board which administers the project. Professor Scholer, head, department of applied mechanics, Kansas State College, Manhattan, is a past-president of ACI.

Following opening ceremonies traffic began rolling on the concrete and asphalt test pavements in a program that will run 6 days a week for the next 2 years. Engineers and technicians of the Highway Research Board's test road staff will record some of the millions of items of data that are expected to influence future highway design and construction.

National Conference on Fundamental Research in Plain Concrete

A highly successful First National Conference on Fundamental Research in Plain Concrete, sponsored by the American Concrete Institute, American Society of Civil Engineers, National Science Foundation, Portland Cement Association, Reinforced Concrete Research Council, and the University of Illinois, was held at the Allerton Park Conference Center of the University of Illinois on Sept. 9-12, 1958.

The conference was attended by 63 participants representing varied fields of interest such as cement chemistry, petrography, and physics from university, government, and industrial laboratories.

The meeting opened September 9 with a welcome from T. J. Dolan, University of Illinois, followed by an introduction to the conference by Clyde E. Kesler, also of the

1959 ACI CONVENTION

February 23-26—Los Angeles

Attention Exhibitors

The local committee for the 55th ACI Annual Convention in Los Angeles, Feb. 23-26, 1959, will sponsor an exhibit in conjunction with the convention.

Firms desiring to exhibit are requested to contact:

Glen Thomas
c/o Southern California Chapter
American Concrete Institute
Room 1058, 742 S. Hill St.
Los Angeles 14, Calif.

University of Illinois. Bryant Mather, Waterways Experiment Station, Jackson, Miss., presided over the first technical session which dealt with the nature, structure, and behavior of concrete. Treval C. Powers, Portland Cement Association, Chicago, was principal speaker at this session. Discussion was limited almost entirely to problems relating to cement pastes constituting a relatively small volume of the concrete, thereby indicating the importance of this portion of the concrete.

Chemistry of concrete

Waldemar C. Hansen, Univesal Atlas Cement Co., Gary, Ind., acted as chairman of the second session devoted to the chemical aspects of concrete, with Kenneth T. Green, Ideal Cement Co., Fort Collins, Colo., serving as speaker for the session. Considerable discussion revolved about techniques of research including x-ray and electron diffraction, differential thermal analysis, infra-red analysis, colloidal techniques, and magnetic resonance. This session made it clear that much research is needed to bridge the gap between the chemical reactions and the physical properties and structure of hardened paste.

Creep and failure considered

The session dealing with mechanical properties and failure, both time independent and time dependent, was presided over by Raymond E. Davis, University of California, Berkeley, with Boris Bresler, University of California, and George W. Washa, University of Wisconsin, Madison, as speakers. In the portion of the session devoted to time independent properties, the discussion was concerned primarily with theories of deformation and failure and the related experiments. Most of the discussion devoted to time dependent properties was concerned primarily with creep. Shrinkage and relaxation were briefly mentioned.

Charles H. Scholer, Kansas State College, Manhattan, led the fourth session which was devoted to failure and deterioration due to other than external load. Richard C. Mielenz, The Master Builders Co., Cleveland, served as principal speaker. Alkali-aggregate reaction was discussed particularly from the standpoint of basic theories of inhibitors to prevent the reaction. Deterioration from freezing and thawing and from sulfate attack was also discussed.

Basic research

Professor Kesler acted as chairman of the fifth session with Gene M. Nordby, University of Arizona, Tucson, speaking on "Importance of Basic Research and Its Support." Hubert Woods, Portland Cement Association, Chicago, discussed possible future conferences, and a summary of the conference and plans for the future were presented by Chester P. Siess, University of Illinois.

Cawdrey to head national contractors group

James W. Cawdrey of Seattle was nominated for president of the Associated General Contractors of America for 1959 at their recent midyear board meeting in Atlanta. Mr. Cawdrey, now vice-president of AGC, is a member of the Seattle firm of Cawdrey and Vemo, building contractors.

John A. Volpe, head of John A. Volpe Construction Co., Malden, Mass., was nominated for the vice-presidency.

Election will be by mail ballot, and new officers will be installed at the 40th annual convention of the organization at Miami Beach in January.

Schaem receives Army award for outstanding achievement

Institute member William E. Schaem, supervising structural engineer in the Office of the Army Chief of Engineers, was among six civilian employees who were presented awards for outstanding achievements in fiscal year 1958 during a special ceremony at the Pentagon on September 25.

Mr. Schaem received a cash award of $7540 for his original proposal for practical and economical method of reinforcing rigid concrete girders with high strength steel bands. To date his suggestion has saved approximately $6 million in construction costs.

This method of strengthening girders was described by Reinhart R. Lunoe and George A. Willis in "Application of Steel Strap Reinforcement to Girders of Rigid Frames, Special AMC Warehouses," ACI JOURNAL, V. 28, No. 7, Jan. 1957 (*Proceedings* V. 53), pp. 669-678.

Loomis establishes consulting firm

Robert S. Loomis has announced the formation of a partnership for the practice of professional engineering in structures and soil mechanics with Raymond H. Loomis under the name of Loomis and Loomis, consulting professional engineers, with offices in Windsor, Conn. Both are members of ACI.

Cohen elected president of Concrete Industry Board

Admiral J. J. Manning, managing director of the Concrete Industry Board, New York, has announced the election of George N. Cohen as president for the ensuing year. Mr. Cohen previously served as vice-president and director of the board. He is currently a vice-president of the Cement League of New York, a division of the New York Building Trades Employers' Association and is an ACI member of many years' standing. Nelson L. Doe assumes the vice-presidency; William J. McIntosh is secretary of the group, and Dugald J. Cameron, treasurer.

The board of directors, chosen from the nine groups comprising every segment of

(*Continued, p. 10*)

LONGEST STRUCTURE on the Illinois Toll Highway is Bridge T-4, which spans the Chicago Sanitary and Ship Canal, the Illinois and Michigan Canal, and the right-of-ways of two railroads just south of LaGrange, Ill. A total of 456 prestressed concrete girders were used to build the two parallel separated lanes of the 4900-ft bridge.

●

Girder span is 88 ft 10¾ in., with a cross section approximately 2 x 4 ft. The 45-ton girders were manufactured by the American Marietta Co. at its Hodgkins, Ill., plant near Chicago, and were delivered ready for installation at the bridge site.

●

Another photograph of this construction by the Illinois State Toll Highway Commission appears on this month's News Letter cover.

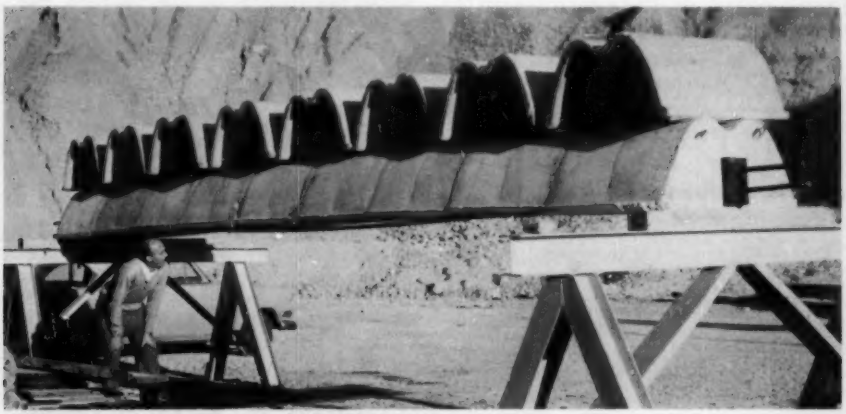
the concrete industry, consists of: Roger H. Corbetta representing the American Concrete Institute; Ingram S. Carner, Architects Council of New York; and Emil Praeger, American Society of Civil Engineers, Metropolitan Section.

The Cement League has three representatives: Bruno Caneva, George P. Dorrance, and H. E. Tear; Dugald J. Cameron represents the Concrete Reinforcing Steel Institute; M. J. McMillan, portland cement manufacturers; Anthony Pope, the ready-mixed concrete producers; and Jesse R. Wingerter, testing laboratories.

Joseph DiStasio, Sr., of DiStasio & Van Buren, was re-elected a director, representing the New York Association of Consulting Engineers, before he met his tragic death on the Jersey Central commuter train which fell into Newark Bay on Monday, September 15.

Rutherford establishes engineering office

Institute member John B. Rutherford of Los Altos, Calif., has opened his own office for the practice of structural engineering. Mr. Rutherford was formerly head of the structural and civil engineering department of Western-Knapp Engineering Co., San Francisco.



Seven shells stressed to form beam . . .

Test load equal to 35 psf was applied to experimental precast, post-stressed shell type concrete beam in a research project undertaken by architecture students at the University of Utah. Individual shell components, measuring about 3 x 3 ft, and 1 ft 7 in. high, reinforced with No. 10 gage steel mesh, were placed end to end with solid end blocks added to form a 27-ft beam. Wave-shaped corrugation of the shells is apparent in the view above, where individual units supplied test load for the beam assembly.

A 1⅛-in. diameter, high strength rod centered below the neutral axis of the assembled beam was stressed to an initial tension of 60,000 psi. Conical steel wedges, slipped over the ends of the rod, and driven on with hydraulic jacks, locked the assembly. Bottom rod shown remained unstressed in this phase of the study. Joints between the units were grouted before the test load was applied.

ACI member Laurence G. Farrant, Salt Lake City consulting engineer, who directed the project, envisages that a number of these shell beam units may be placed side by side and connected to form a complete roof structure. He estimates that a comparable unit designed to span 200 ft should be cast 8 ft high, as compared to the 1 ft 7 in. height of the 27-ft prototype.

LACLEDE HIGH QUALITY REINFORCING STEELS ADD MUSCLE TO NEW EXPRESSWAY BRIDGE

This new highway 40 overpass over the Mark Twain Expressway is an integral part of a road building program designed to speed traffic through suburban St. Louis.

As in many road building projects, Laclede high strength reinforcing steels fortify this bridge for heavy traffic—assuring durability and strength.

Use these Laclede highway steels to put muscle in your construction projects:

Welded wire fabric
Welded dowel spacers
Prestressed strand
Multi-rib round reinforcing bars
Center joints
Recess joints
Tie bars
Accessories

General Contractor: Fruin-Colnon Contracting Company, St. Louis

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FIRST NEW HIGHWAY-ACT PAVEMENT . . . BUILT IN 1957

POZZOLITH* improved concrete

recommended with confidence for projects like this . . .

. . . because of performance on projects like this . . .

Above: Section of U. S. Route 40, west of Topeka. Kansas State Highway Commission. Contractor: Koss Construction Co.

Below: Route 5, approximately 5 miles east of Erie. Pennsylvania State Highway Department. Contractor: Geo. S. Mellert-Weidner Company.

BUILT IN 1942 . . .

. . . concrete is in exceptionally good condition today. Another project where superior durability was obtained economically with POZZOLITH-improved concrete.

*POZZOLITH . . . registered trademark of The Master Builders Company for its water-reducing, air-entraining admixture for concrete.

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Position open for materials engineer in Seattle

The Bureau of Yards and Docks, Department of the Navy, announces a vacancy at its Seattle field office for the position of director of the material testing and evaluation division, Thirteenth Naval District. Civil engineering degree or experience equivalent required, plus 3½ years of progressive professional engineering experience of which one year must be in the specialized field of engineering which entails research, analysis, or development of materials.

Salary range is from $8810 to $9530 per year. For further details contact: Industrial Relations Officer, District Public Works Office, Thirteenth Naval District, Building 250, U. S. Naval Station, Seattle 99, Wash.

Fromme and Vosganian form consulting firm

Alvin Fromme and Zorab Vosganian have announced the formation of a partnership to be known as Fromme and Vosganian, consulting engineers, with offices in New York. They will specialize in the structural design of timber, structural steel, reinforced concrete structures, and foundations.

New ASTM technical committee officers

A number of ACI members have recently been added to the roster of ASTM technical committees. Among the newly appointed officers are W. J. McCoy, Lehigh Portland Cement Co., Allentown, Pa., who will serve as secretary of Committee C-1 on Cement, and Dalton G. Miller, University of Minnesota, St. Paul, who will assume the chairmanship of Committee C-4 on Clay Pipe.

Hardesty and Hanover admit three new partners

Clinton D. Hanover, Jr., has announced that Henry W. Fischer, Egbert R. Hardesty, and John Fiala, who have long been associated with the firm of Hardesty and Hanover, New York, have been admitted to the firm as partners.

Settoon becomes associate of New Orleans firm

Announcement has been made of the appointment of William B. Settoon, civil engineer, as an associate of the firm of Curtis and Davis, architect-engineers, New Orleans. Mr. Settoon has been an ACI member since 1953.

LOOKING AHEAD

Dec. 7-10, 1958—American Institute of Chemical Engineers, Annual Meeting, Netherland Plaza Hotel, Cincinnati, Ohio

Dec. 10-11, 1958—National Construction Industry Conference, Fourth Annual Meeting, Sherman Hotel, Chicago, Ill.

Jan. 12-15, 1959—39th Annual National Concrete Masonry Association Convention and 11th Concrete Industries Exposition, Cleveland Public Auditorium, Cleveland, Ohio

Jan. 25-29, 1959—Associated Equipment Distributors, 40th Annual Meeting, Conrad Hilton Hotel, Chicago, Ill.

Jan. 27-30, 1959—National Crushed Stone Association, 42nd Annual Convention, Bal Harbour Area Hotel Group, Miami Beach, Fla.

Feb. 2-6, 1959—American Society for Testing Materials, Committee Week, William Penn Hotel, Pittsburgh

Feb. 15-19, 1959—National Sand and Gravel Association, 43rd Annual Convention, Hotel Roosevelt, New Orleans, La.

Feb. 23-26, 1959—American Concrete Institute, 55th Annual Convention, Statler Hilton Hotel, Los Angeles, Calif.

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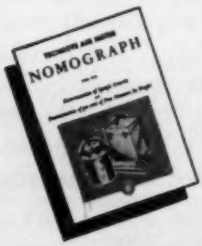
FIRST CHOICE AROUND THE WORLD

The Techkote Air Meter now provides a multi-range testing instrument that vastly increases the usefulness of the air meter. It is now possible to measure accurately—within a fraction of a degree—entrained air with one instrument on numerous materials such as lightweight concrete, mortar, plaster and soil, in addition to regular concrete.



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Who's Who This Month

ACI Committee 605

"Proposed ACI Standard: Recommended Practice for Hot Weather Concreting," prepared by ACI Committee 605, Hot Weather Concreting, appears on p. 525. Stanton Walker is chairman of this committee, with Delmar L. Bloem serving as secretary. Both Mr. Walker and Mr. Bloem are affiliated with the National Ready Mixed Concrete Association and National Sand and Gravel Association, Washington, D. C.

Members of the committee are: Clayton L. Davis, Universal Atlas Cement Division, United States Steel Corp., New York; John L. Goetz, Southwestern Portland Cement Co., Los Angeles; Nomer Gray, Ammann and Whitney, New York; Ernst Gruenwald, Lone Star Cement Corp., New York; and H. F. Hedderich, Pittsburgh Testing Laboratory, Pittsburgh.

E. L. Howard, Pacific Cement and Aggregates, Inc., San Francisco; Harry F. Irwin, Warner Co., Philadelphia; Harry Mitchell, Spartanburg Concrete Co., Spartanburg, S. C.; George H. Nelson, Law Engineering Testing Co., Atlanta; and George L. Otterson, consultant, Chicago, are also members of Committee 605.

Rounding out the 20-man committee are Perry H. Petersen, The Master Builders Co., Cleveland; Emil Schmid, Sika Chemical Corp., Passaic, N. J.; John H. Swanberg, Minnesota Department of Highways, St. Paul; and Lewis H. Tuthill, California State Department of Water Resources, Sacramento.

Also I. L. Tyler, Portland Cement Association, Chicago; Byron P. Weintz, Consolidated Rock Products Co., Los Angeles; John J. White, Parsons, Brinckerhoff, Hall and MacDonald, New York; and C. E. Wuerpel, Marquette Cement Manufacturing Co., Chicago.

ACI Committee 614

Report of ACI Committee 614 on "Proposed Revision of ACI Standard 614-42: Recommended Practice for Measuring, Mixing, and Placing Concrete" appears on p. 535.

Lewis H. Tuthill, California State Department of Water Resources, Sacramento, is chairman of Committee 614 and was assisted in preparing the report by the following 12-man committee: A. W. Brust, Washington University, St. Louis; H. F. Hedderich, Pittsburgh Testing Laboratory, Pittsburgh; E. L. Howard, Pacific Cement & Aggregates, Inc., San Francisco; J. J. Manning, The Concrete Industry Board, New York, and H. H. McLean, New York Department of Public Works, Albany.

Also W. T. Neelands, Sverdrup and Parcel Engineering Co., Bangkok, Thailand; J. A. Nicholson, Nicholson Concrete Co., Toledo; Bert Noble, Noble Co., Oakland, Calif.; J. W. Poulter, Doehring Co., Milwaukee; T. J. Reading, Corps of Engineers, Omaha; M. R. Smith, Office Chief of Engineers, Department of the Army, Washington, D. C.; J. W. Winkworth, Winkworth Fuel and Supply Co., Detroit; L. P. Witte, Bureau of Reclamation, Denver; and D. K. Woodin, East Bay Municipal Utility District, Oakland, Calif.

International Council for Building Research

"Load Factors" on p. 567 represents a report submitted to the International Council for Building Research, Paris, France. Eduardo Torroja, director, Instituto Tecnico de la Construccion y del Cemento, Costillares (Madrid), Spain, served as chairman of the committee which rendered this report.

The committee was comprised of: Nicolas Esquillan, technical director, Boussiron Enterprises, Paris, France; J. P. Mazure, professor, Institute of Technology, Delft, Holland; Giuseppe Rinaldi, chief civil engineer, Bologna, Italy; Hubert Rüsch, professor and director of laboratory for testing building materials and structures, Institute of Technology, Munich, Germany; and Frederick G. Thomas, civil engineer, Building Research Station, Garston, Watford, Herts, England.

add extra years of life to concrete with
CLINTON WELDED WIRE FABRIC

Experienced contractors know that fabric-reinforced concrete lasts longer. And over the years it looks better and requires much less maintenance than unreinforced concrete. Fabric-reinforcement gives these advantages because it adds the tensile strength of steel to concrete . . . resists the heaving and cracking caused by temperature extremes and moisture content . . . minimizes cracking during setting . . . and distributes stress evenly in all directions.

Even if a crack does develop, the fabric holds it tightly together, preserves the smooth surface, and retards further lengthening of the crack. No other type of reinforcement is so easy to install, yet gives so much extra strength for so little cost.

Clinton Welded Wire Fabric is a top-quality product that is readily available in both East and West, in a wide variety of gages, lengths and widths. On your next job, add the tensile strength of steel with Clinton Welded Wire Fabric, and rest assured that you've built a concrete surface that will last longer with minimum maintenance.

WHEN THEY ASK...
"is it Reinforced"
SAY YES...WITH

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THE COLORADO FUEL AND IRON CORPORATION

THE COLORADO FUEL AND IRON CORPORATION—Albuquerque • Amarillo • Billings • Boise • Butte • Denver • El Paso • Ft. Worth Houston • Kansas City • Lincoln • Los Angeles • Oakland • Oklahoma City • Phoenix • Portland • Pueblo • Salt Lake City San Francisco • San Leandro • Seattle • Spokane • Wichita • WICKWIRE SPENCER STEEL DIVISION—Atlanta • Boston • Buffalo Chicago • Detroit • New Orleans • New York • Philadelphia • CF&I OFFICES IN CANADA: Montreal • Toronto
CANADIAN REPRESENTATIVES AT: Calgary • Edmonton • Vancouver • Winnipeg

6085

George C. Ernst

The research paper on "Moment and Shear Redistribution in Two-Span Continuous Reinforced Concrete Beams" appearing on p. 573 was written by George C. Ernst, professor of civil engineering and director of the Engineering Experiment Station at the University of Nebraska, Lincoln.

Professor Ernst graduated from the University of Michigan in 1929 and has been in academic work since 1930 at Iowa State College, University of Maryland, and University of Nebraska. In 1956 he resigned from the chairmanship of the civil engineering department at the University of Nebraska to devote full time to teaching and research. Professor Ernst was awarded the CE degree from the University of Michigan in June 1958.

Professor Ernst is a 1958 nominee to the ACI Board of Direction. He has been an Institute member since 1935 and has actively participated in committee work and contributed generously to the pages of the JOURNAL. He is currently chairman of the newly organized ACI Committee 338, Torsion, and a member of Committee 115, Research.

F. A. Blakey

"Influence of Water-Cement Ratio on Mortar in Which Shrinkage Is Restrained" appearing on p. 591 was submitted by F. A. Blakey, officer-in-charge of the concrete and testing laboratory, Division of Building Research, Commonwealth Scientific and Industrial Research Organization, Melbourne, Australia.

Dr. Blakey graduated in 1945 from the University of Western Australia, Perth, with first-class honors in structural engineering. After 2 years as assistant lecturer in civil engineering at the University of Tasmania, Hobart, he was awarded the Hackett Traveling Studentship and went to Cambridge University where he received his PhD in 1949.

Dr. Blakey was coauthor with I. Leber of "Some Effects of Carbon Dioxide on Mortars and Concrete," published in the September 1956 JOURNAL.

Jack R. Benjamin and Harry A. Williams

Coauthors Jack R. Benjamin and Harry A. Williams present "Behavior of One-Story Reinforced Concrete Shear Walls Containing Openings," on p. 605 of this issue.

Jack R. Benjamin, an ACI member, is associate professor of structural engineering at Stanford University, Stanford, Calif. He received his ScD from the Massachusetts Institute of Technology in 1942 after obtaining his BS and MS in civil engineering at the University of Washington.

Following service in the army, Professor Benjamin was assistant professor of structural design at Rensselaer Polytechnic Institute from 1946 to 1948 prior to joining the faculty at Stanford University.

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Harry A. Williams is professor of civil engineering at Stanford University. He graduated from Stanford in 1925 and was affiliated with the engineering department of the Standard Oil Co. of California for the next 5½ years, first as a draftsman and later progressing to the position of designing engineer in charge of various refinery projects.

Professor Williams returned to Stanford in 1930 and obtained his CE degree while teaching part time. He became a full-time instructor in 1933 and has remained at Stanford since then, teaching a variety of subjects but specializing primarily in applied mechanics and structures.

In addition to various consulting engagements Professor Williams has spent sabbatical leaves with the Corps of Engineers, the

Bureau of Reclamation, and Massachusetts Institute of Technology. Since 1946, he has conducted research spnsored by the National Advisory Council for Aeronautics, the Air Force, Corps of Engineers, and the Armed Forces Special Weapons Projects. This and other research have resulted in a number of publications.

Professor Williams is currently a member of ACI Committee 115, Research, and Committee 335, Deflection of Concrete Building Structures.

D. W. Lewis

"Lightweight Concrete Made with Expanded Blast Furnace Slag," p. 619, was presented at the 1958 ACI Regional Meeting in Detroit last month by the author, D. W. Lewis.

Mr. Lewis is chief engineer of the National Slag Association, Washington, D. C. He graduated from the University of Wyoming in 1941 and obtained an MS degree in civil engineering from Purdue University in 1942.

In 1946, following Army service in World War II, Mr. Lewis joined the staff of Purdue University. During the next 9 years he supervised research work in concrete and concrete aggregates as research engineer, Joint Highway Research Project, and also taught in the School of Civil Engineering. He joined the National Slag Association in his present capacity in 1955.

Mr. Lewis is a member of ACI Committee 116, Nomenclature; Committee 201, Durability of Concrete in Service; and Committee 716, High Pressure Steam Curing. He is the author or coauthor of a number of papers in the field of concrete testing, particularly from the standpoint of durability, published by ASTM, Highway Research Board, and other organizations.

J. W. Baldwin, Jr. and Ivan M. Viest

"Effect of Axial Compression on Shear Strength of Reinforced Concrete Frame Members" on p. 635 is the joint contribution of J. W. Baldwin, Jr., and Ivan M. Viest. The authors originally presented their paper at the 54th annual ACI convention in Chicago last February.

J. W. Baldwin, Jr., instructor in theoretical and applied mechanics, University of Illinois, received his BS in general engineering from the University of Illinois in 1951. After serving 3½ years in the U. S. Army Corps of Engineers, Mr. Baldwin returned to the University of Illinois to do graduate work, receiving his MS in 1956. His master's thesis pertained to part of the work presented in this paper. Engaged in research in the field of reinforced concrete for the past 3 years, Mr. Baldwin is presently working on his PhD in the field of brittle materials.

Ivan M. Viest, bridge research engineer, AASHO Road Test, Ottawa, Ill., graduated from the Slovak Technical University, Bratislava, Czechoslovakia, in 1946; he received his MS degree in civil engineering from the Georgia Institute of Technology in 1948 and his PhD in engineering from the University of Illinois in 1951.

Since 1948 his work has been devoted to several research projects pertaining to steel and reinforced concrete structures. In 1955 he was awarded the ACI Wason Medal for Research, along with his collaborators, for work reported in their four-part paper entitled "Shear Strength of Reinforced Concrete Beams." In 1958 he received the ASCE Research Award. He has been active as consultant on various problems of industrial development and research, and prior to assuming his present position was research associate professor at the University of Illinois.

Active in ACI technical committee work for many years, Mr. Viest is currently chairman of ACI-ASCE Committee 333, Design and Construction of Composite Structures, and a member of ACI-ASCE Committee 326, Shear and Diagonal Tension.

Bloomfield heads modular association

The recently formed Modular Building Standards Association has named Byron C. Bloomfield to the position of executive director. Mr. Bloomfield was formerly secretary for professional development with the American Institute of Architects.

MBSA was formed in 1957 under the joint sponsorship of the American Institute of

Membership in the American Concrete Institute

To facilitate prospective members in joining the Institute, membership application forms are provided. Present Members may aid by bringing these forms to the attention of those who may profit from membership advantages. The grades of membership are described overleaf.

All who have an interest in concrete are eligible for membership.

Members have at hand in Institute publications the most complete fund of knowledge on concrete. The ACI JOURNAL provides them with the latest information and ACI special publications provide them with the complete picture of specific problems. Through conventions, and regional and area meetings they are afforded the opportunity of meeting those whose experiences provide the new information, and of exchanging ideas with them.

Opportunity for service is present in technical committee activity, in contributions, or only comments, to the ACI JOURNAL, or in reviewing technical publications for material of interest to the membership.

ACI's world-wide membership is growing in extent and participation—traveling a common road toward better, more economical and durable concrete structures. ACI provides a common ground in the search for and use of new "working tools" in concrete design, manufacture, and erection—and its interpretation.

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EXCERPTS FROM BYLAWS: ARTICLE I—MEMBERS

Section 1. This Institute shall consist of Honorary Members, Corporation Members, Contributing Members, Members, Junior Members and Student Members.

Sec. 3. A Member shall be a person.

A Corporation Member shall be a firm, corporation, society, agency of government, or other organization.

A Contributing Member shall be a person, firm, corporation, society, agency or government, or other organization electing to give greater support to Institute activities through the payment of larger dues. Any Contributing or Corporation Member, other than a person, may name a personal representative who shall enjoy all membership rights and privileges.

A Junior Member shall be a person less than 28 years of age.

A Student Member shall be a person less than 28 years of age and a registered student at a technical or engineering school.

Sec. 4. All classes of Members, except Honorary Members and Student Members, shall be sponsored by at least one Member of the Institute. An Honorary Member shall be elected by unanimous vote of the Board of Direction. A Student Member shall be sponsored either by a Member of the Institute or by a member of his school's faculty, who need not be a Member of the Institute.

Sec. 5. All Members in any classification shall have all rights and privileges of membership as determined by the Board of Direction except that a Junior or Student Member shall neither vote nor hold office. The status of a Student Member shall change automatically to that of Junior Member or Member, depending on age, on the first anniversay of his membership succeeding the date on which he ceases to be a registered student. The status of a Junior Member shall be changed to that of Member on the first anniversary of his membership after he becomes 28 years of age.

Sec. 6. Applications for and resignations from membership and requests for change of representatives of Corporation or Contributing Members shall be presented in writing to the Secretary-Treasurer. Resignations may be accepted only from Members whose dues are not more than 60 days in arrears, except by special action of the Board of Direction.

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Architects, Associated General Contractors, National Association of Home Builders, and the Producers' Council, Inc. The organization's headquarters are in Washington, D. C.

BRI to sponsor paint symposium

The Building Research Institute will present a "paint progress" conference on December 3 and 4 at the Shoreham Hotel, Washington, D. C.

Twenty of the nation's top authorities on today's new paints and protective coatings will participate in the program. Special reports will be given on new paints and coatings for masonry and concrete substrates.

The meeting is open to the public as well as BRI members and guests. The full program and further information about the conference are available from Harold Horowitz, BRI technical secretary, 2101 Constitution Ave., Washington 25, D. C.

Lehigh's Browning to retire

The board of directors of Lehigh Portland Cement Co., Allentown, Pa., have announced the retirement of C. Glenn Browning, executive vice-president in charge of sales, effective Dec. 31, 1958. Mr. Browning has been with Lehigh for 37 years, and will continue to serve as a director of the company.

Ralph L. Browning, who has served as vice-president in charge of sales for the past 6 years, will succeed C. Glenn Browning as head of the sales division.

Alpha appoints Ziegler assistant to vice-president

Daniel L. Ziegler has been appointed assistant to the vice-president in charge of operations at Alpha Portland Cement Co., Easton, Pa. He will work out of the home office in Easton.

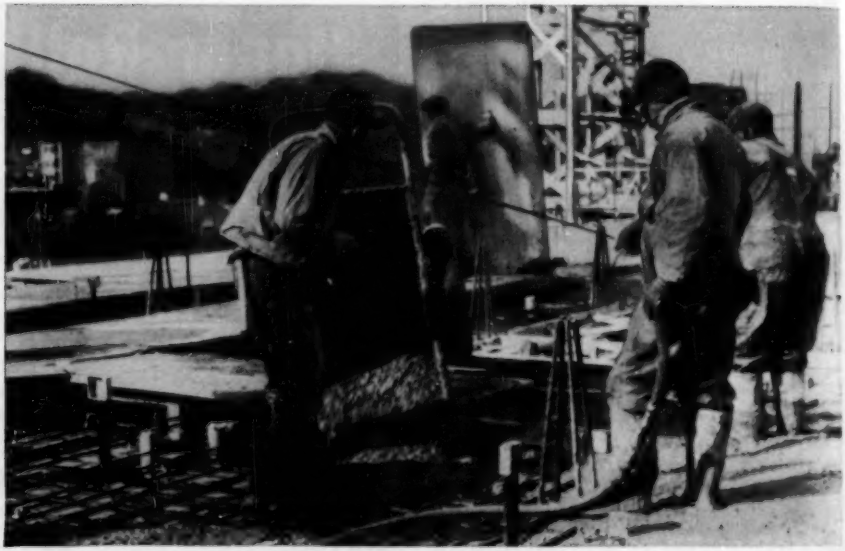
Prior to his promotion, Mr. Ziegler was company project manager. Much of his work since joining Alpha as plant engineer at the Catskill, N. Y., plant in 1922 has been concerned with the building and planning of improvements at the company's plants. He supervised construction of the firm's recently completed cement plant at Lime Kiln, Md.

Honor Roll

January 1—October 31, 1958

Heading the Honor Roll list appears conducive to being "concrete conscious" or is it personal enterprise in meeting the responsibilities of an organization founded on mutual effort? Blas Lamberti has retained the edge this month in stressing the importance of ACI with credit for 15 new members. Joseph J. Waddell is still second with 13 approved applicants.

Blas Lamberti	15
Joseph J. Waddell	13
Phil M. Ferguson	10
L. M. Legatski	7
James A. McCarthy	7
Ernst Maag	6
H. C. Pfannkuche	6
Jerome M. Raphael	6
J. P. Thompson	6
Samuel Hobbs	5
Carl H. Koontz	5
Douglas McHenry	5
C. P. Siess	5
Ernest L. Spencer	5
Robert P. Witt	5
Moises Bendahan	4
Celso A. Carbonell	4
Jaime de las Casas	4
P. E. Ellen	4
N. J. Everard	4
Martin J. Gutzwiller	4
E. L. Howard	4
Frank E. Legg, Jr.	4
John B. Porter	4
Walter H. Price	4
C. H. Scholer	4
Clarence A. Walkwitz	4
Luther E. Bell	3
Vincent R. Cartelli	3
M. A. Craven	3
Carl E. Ekberg, Jr.	3
Aleck S. Evans	3
Richard W. Gunn	3
Paul A. Hansen	3
Robert B. Harris	3
A. T. Hersey	3
Kenneth M. Huber	3
E. Richard Kummerle	3
Daniel S. Ling	3
George A. Mansfield	3
H. P. Mittet	3
E. Schmid	3
Howard Simpson	3
Otis D. Small	3
O. C. Struthers	3
T. W. Thomas	3
Ellis S. Vieser	3
Byron P. Weintz	3
William M. Avery	2
Robert B. Banning	2



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The American Concrete Institute now recommends the addition of calcium chloride for cold weather concreting (ACI Standard "Recommended Practice for Winter Concreting"). All this evidence should suggest a check with your ready mix suppliers or a new look at your own batching procedures. If your files aren't up-to-date, a note to our Pittsburgh address or to any of the fourteen Columbia-Southern District Sales Offices will get you free copies of helpful booklets like *Better Concrete All Year Round* or *Columbia Calcium Chloride In Concrete.*

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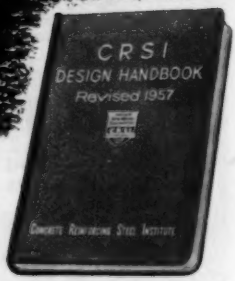

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New Members

The Board of Direction approved 61 Individual applications, 1 Corporation, 15 Junior, and 7 Student applications making a total of 84 new members. Considering losses due to deaths, resignations, and nonpayment of dues the total membership on Oct. 1, 1958 was 9622.

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ATLAS, AMOS, New York, N. Y. (Design Engr., Ammann & Whitney)
BAY, ROBERT DEWEY, St. Louis, Mo. (Sales Engr., Laclede Steel Co.)
BERNANDER, KARL-GUSTAV, Stockholm-Bandhagen Sweden (C. E., Head of Tech. Dept., A. B. Strangbetong)
BROWN, CAPT. LEVI A., Grosse Pointe Farms, Mich. (Capt., Corps. of Engrs., Dept. of the Army, Washington, D. C.)
BUFFENMYER, DARL T., Levittown, Pa. (Tech. Service Engr., Sika Chemical Corp.)
CHENEY, LLOYD T., Detroit, Mich. (Assoc. Prof. of Civil Engrg., Wayne State Univ.)
CLEVELY, JAMES WILLIAM, Feilding, New Zealand (Asst. Engr. to Board, Manawatu-oroua Electric Power Board)
COTTON, LAWRENCE J., Oklahoma City, Okla. (Owner, Southwest Concrete Products)
DE LA BRUNIERE, BERNARD, New York, N. Y. (Tech. Adviser, Pontex Pipe Corp.)
DEUTSCH, JOSEPH A., Montebello, Calif. (Owner, J. A. Deutsch & Co., Gen. Contr.)
DICKINSON, H. J., Denver, Colo. (Chf. Constr. Engr., Tipton & Kalmbach, Inc.)
DLUBACZ, RICHARD, Detroit, Mich. (Tech. Dept., Huron Portland Cement Co.)
DOBELL, CURZON, Nassau, Bahamas (Cons. Engr.)
ERLIJMAN, HORACIO, San Francisco, Calif. (Design Engr., International Engrg. Co.)
FAIRLIE, JAMES MCLACHLAN, Stockton-on-Tees Co., Durham, England (Concrete Engr.)
FENG, CHUAN CHUNG, University, Miss. (Assoc. Prof. of Civil Engrg., Univ. of Miss.)
FLOYD, BEN H., Oklahoma City, Okla. (Partner, McDonald & Floyd, Cons. Engrg. Service)
FORNI, JAY J., Emeryville, Calif. (Mfr. of all types of precast concrete & prestressed products, Geo. P. Forni, Inc.)
FREEMAN, ROBERT L., Detroit, Mich. (Sales Mgr. & Tech. Dir., National Ready Mix, Inc.)
FROST, CARLTON S., St. Petersburg, Fla. (Pres., Gulf Coast Testing Lab., Inc.)
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GRAVES, WALTER T., Minneapolis, Minn. (Assoc. Prof. of Civil Engrg., Univ. of Minn.)
GRISA, FRANK L., Hales Corners, Wis. (C. E., West Allis Concrete Products Co.)
GUISE, ROBERT F., JR., Ann Arbor, Mich. (Chf. Engr., O. F. Goodwin Contracting & Gravel Co.)
HAPKE, DANIEL B., Oklahoma City, Okla. (Struct. Engr., Field Promotion, PCA)
HELVIE, PAUL D., Princeton, Ky. (Chf. Engr., Wasson Coal Mining Corp.)
HEWS, RICHARD J., Yakima, Wash. (Pres. & Mgr., Yakima Cement Products Co.)
HODSON, RICHARD C., Long Beach, Calif. (Sales Engr., L. M. Scofield Co.)
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HOSKINS, N. G., Walkerville, South Australia (Mgr. of Adelaide Office, R. Crooks, Michell & Peacock)
IBARRA, CLEMENTE SANZ, Col. Unidad Modelo, Mexico (Mgr. of Structures, DIRAC)
IERUBINO, ANTON T., Newark, N. J. (Struct. Design Engr., Boswell Engrg. Co.)
IRUJO, JUAN IGNACIO, Caracas, Venezuela (C. E., Supv., Insp., Designing, Ministerio de Obras Publicas)
IUANIER, MAXIMILIANO, Montreal, Que., Canada (Engr., Ross, Fish, Duschenes & Barrett, Archs. & Engrs.)
KREPS, ROBERT R., Toledo, Ohio (Struct. Engr., Raymond C. Reese)
LOCKETTE, PHILIP C., Glenrock, Wyo. (Office Engr., Ebasco Services, Inc.)
MCDONALD, DONALD M., Fresno, Calif. (District Testing Engr., Pacific Cement & Agg., Inc.)
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MELL, FRANCIS S., Akron, Ohio (Salesman, The Fairlawn Supply Concrete Co.)
MINOR, WESLEY L., Granada Hills, Calif. (Area Repres. & Supt. of Applications, California Zonolite Co.)
NIEBER, ALLEN J., JR., Livonia, Mich. (Design Engr., Price Bros. Co. (Dayton, Ohio), Michigan Flexicore Div.)
OGREN, P. H., Chicago, Ill. (Mgr., Constr. Equip. Div., Wyzenbeek & Staff, Inc.)
O'NEIL, J. E., Wellesley Hills, Mass. (Research & Matls. Engr., Commonwealth of Mass., DPW)
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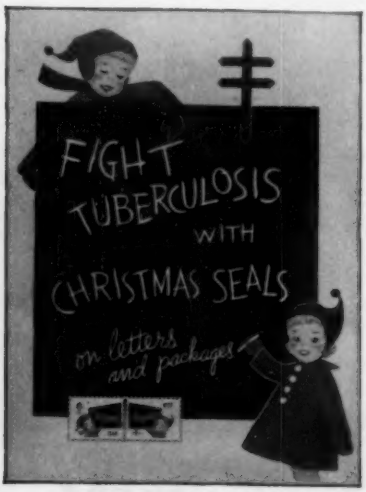
TOENNIES, HENRY, Elmhurst, Ill. (Dev. Engr., National Concrete Masonry Ass'n.)



VINSON, L. G., Phoenix, Arizona (Pres. & Gen. Mgr., Vinson Construction Co.)
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SANDERSON, IAN, Whakatane, Bay of Plenty, New Zealand (Company Draughtsman, F. G. Mahy & Co., Ltd.)
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1. The names and addresses of the publisher, editor, managing editor, and business managers are:

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100 YEARS OF QUALITY PRODUCTS . . . 1858-1958

Tools, Materials, Services

Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

Plastic interior silo coating

An interior silo coating, Dura-a-Cote, developed by Marietta Concrete Corp. in cooperation with Bakelite Co., reportedly will not crack, chip or peel, and seals all the pores and joints in a silo wall. The epoxy material is mixed with the hardener at the silo site and hand troweled onto the inside wall; it air-dries, requiring no heat to harden into an air-tight finish, resistant to most heavy blows or scraping from forks and unloaders.

Marietta further states that Dur-A-Cote is not affected by corrosive agents in silage such as acetic, lactic, and butyric acids; also the smooth surface promotes packing of silage and its tight seal preserves nutritious elements and reduces spoilage.—Bakelite Co., Division of Union Carbide Corp., 655 Madison, New York 21, N. Y.

Twin roll crusher with hydraulic adjustment

Pioneer Engineering has recently developed a twin roll crusher, Model 4030, for aggregate producers, which embodies design features intended to cut maintenance costs and to provide considerably higher capacities of controlled sizes of product, according to the company. Increased capacity results from a 35 percent increase in width of roll face, and a rim speed approximately 10 percent faster.

Model 4030 is available with two smooth shells and will take a feed up to 4 in.; it has hydraulic adjustment for crusher setting, and power required ranges from 135 to 160 hp depending on the size of material to be produced and the degree of toughness of the feed.

The crusher measures approximately 5½ ft high and 14 ft long; 8½ to 9½ ft is overall width, depending on the type of drive shaft desired. Total weight, approximately 37,000 lb.—Pioneer Engineering, Division of Poor and Co., Inc., 3200 Como Ave., Minneapolis 14, Minn.

Grease removing agent for concrete

Oil or grease soaked concrete floors can be restored to clean condition through the use of Greasup according to the Berylex National Sales organisation. The product is described as a powerful chemical emulsion which cleans concrete by deep penetration, dissolving all greases, oil, fats, waxes, etc. below the surface. Also it is safe to use, having a high flash point and is low in toxicity.

When a grease soaked concrete surface is to be retopped or painted, the Berylex company recommends a two step treatment, following the application of Greasup with their product, Alprep, a phosphoric acid type of cleaner, which opens the pores of the concrete, etches and neutralizes the cleaned surface, permitting good bond for new topping or surface painting.—Berylex National Sales, Division of Harry Wardo and Co., Inc., Box 33, Rosedale Station, Kansas City 3, Kans.

Air placement method

Airplaco concrete placers are available in two models: Model CP-10 which handles 10 cu ft of concrete per charge and Model CP-15 handling 15 cu ft per charge. Either 4 in. or 6 in. inside diameter concrete placement tubing is available for use with both models. According to the manufacturer these models will place concrete with 2 in. to 6 in. slump, and can handle concrete made with harsh aggregate such as Haydite. Production rates range from 8 to 20 cu yd per hr depending on the slump of the concrete, length of placement tubing, vertical lift, and air supply.

With this placement method concrete is discharged from either a standard discharge box or from special discharge sections designed to fit narrow forms, etc. Company spokesman reports important features of these models include complete portability to and from job site and on-the-job maneuverability. Special couplings on each section of concrete placement tubing assure speed and flexibility in set-up.—Air Placement Equipment Co., 1009 West 24th St., Kansas City 8, Mo.

Literature Available

Pertinent details on the latest equipment and products on the market are available in recently released literature. Exact titles of the booklets and catalogs are indicated in capital letters. They may be requested direcly from the manufacturers listed below.

HI-BOY TRUCK MIXERS (Bulletin No. 2622)—Specifications and illustrations of Hi-Boy Truck Mixers (5, 6, and 7 cu yd models) and their component parts are highlighted in 16-page catalog.—Blaw-Knox Co., Sales Promotion Dept., Mattoon, Ill.

PRESTRESSED CONCRETE DESIGN HANDBOOK, V. 9—This volume covering Leap 18-in. double-T prestressed concrete roof members (5 ft wide flange) may be purchased for $2.00. Handbook contains complete technical information on 116 different combinations which will enable an architect or engineer to specify exactly which double-T to use under any given condition. Data included cover safe superimposed loads in the span range of 20 ft through 61 ft using 7/16 in. diameter strand with both parallel and depressed designs. Data sheet for each strand pattern gives spans, loadings, prestressing force, initial prestressing force, principal concrete fiber stresses, camber, deflection, release strength of concrete, and 2-day concrete strength. Also includes section on basis of design of prestressed concrete and table of loadings for nine other prestressed structural members such as the channel and keystone joist.—Leap Concrete, Inc., P. O. Box 1053, Lakeland, Fla.

POZZOLITH EMPLOYED IN LIGHTWEIGHT CONCRETE (Master Builders Reporter No. 14)—Recently published 20-page brochure illustrates the wide use of lightweight aggregate concrete in modern construction. Photograph and job stories present a clear picture of the wide variety of uses for this relatively new and versatile building material.—The Master Builders Co., 7016 Euclid Ave., Cleveland 3, Ohio.

1958 INDUSTRY DIRECTORY—Associated Equipment Distributors announce the publication of their 1958 directory. The 382-p. volume is divided into four sections: Distributors, Manufacturers, Products, and Trade Names. Lists company names, addresses, key personnel, and phone numbers for over 2000 firms in the construction industry. $20.00 per copy.—Associated Equipment Distributors, 30 East Cedar St., Chicago 11, Ill.

STANDARD CALCIUM CHLORIDE SOLUTION IN READY MIXED CONCRETE—Pamphlet designed especially for ready-mixed concrete users in their preparation and use of calcium chloride standard solution. Tells how to prepare the standard solution, the amount to use per batch of concrete, and provides data on automatic dispensers.—Calcium Chloride Institute, 909 Ring Building, Washington 6, D. C.

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With a "Special Winterized" mix containing 2% of SOLVAY Calcium Chloride, heated water and aggregate, you keep close to warm weather schedules at any temperature—get concrete with 8 to 12% greater ultimate strength . . . more workability. This permits lower water-cement ratio, resulting in denser and more moisture- and wear-resistant concrete.

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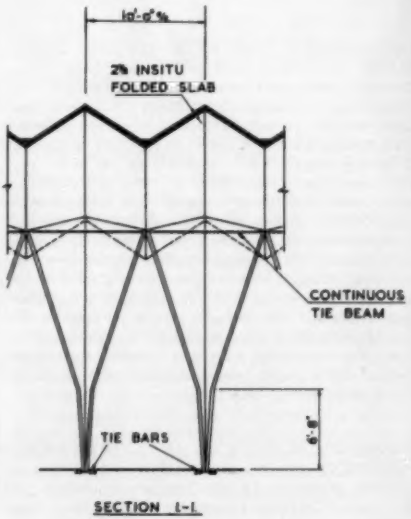
SOLVAY dealers and branch offices are located in major centers from coast to coast.

HOW TO LAY HEAVY-DUTY, HEAT-RESISTANT FLOORING—Bulletin HR-2 describes four-step process for laying heavy-duty, heat-resistant flooring. Illustrations supplement the description of materials and steps used in laying Emeri-Brick floors. Cost and time saving factors discussed; dimensions and specifications of Emeri-Brick included.—Walter Maguire Co., Inc., 60 East 42nd St., New York 17, N. Y.

BULLDOG GOLD DIGGER CONCRETE DRILL ANCHORS (Catalog 58)—Five types of drill-anchors are cataloged including special flush, rod hanger, tie wire, and stud-head anchors (installation with electric or air hammer) and a flush anchor installed by hand hammer only. Complete specifying information as well as installation-application data included.—Polis Manufacturing Co., Division of Gregory Industries, Toledo Ave. and East 28th, Lorain, Ohio

Addendum to "Folded Slab Construction"

The accompanying illustrations represent an addition to the paper "Folded Slab Construction," by Felix J. Samuely, which appeared in the October 1958 ACI JOURNAL. These drawings show two cross sections of the assembly hall described by Mr. Samuely at the bottom of p. 459. The folded slab not only forms the roof in this construction, but it is carried around the side elevation and in fact replaces a rigid frame. Because of headroom it is carried down only to a height of 6 ft 8 in., with the vertical portion of the frame carried through to the bottom, and made rigid enough to take the bending moments caused by the horizontal thrust.



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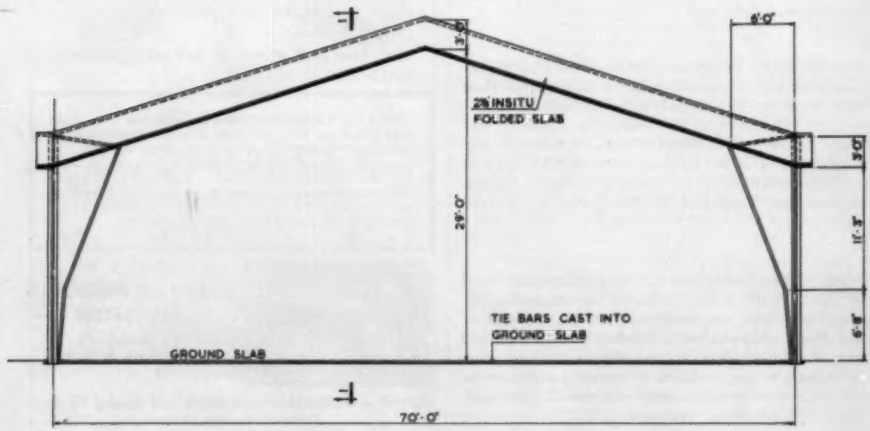

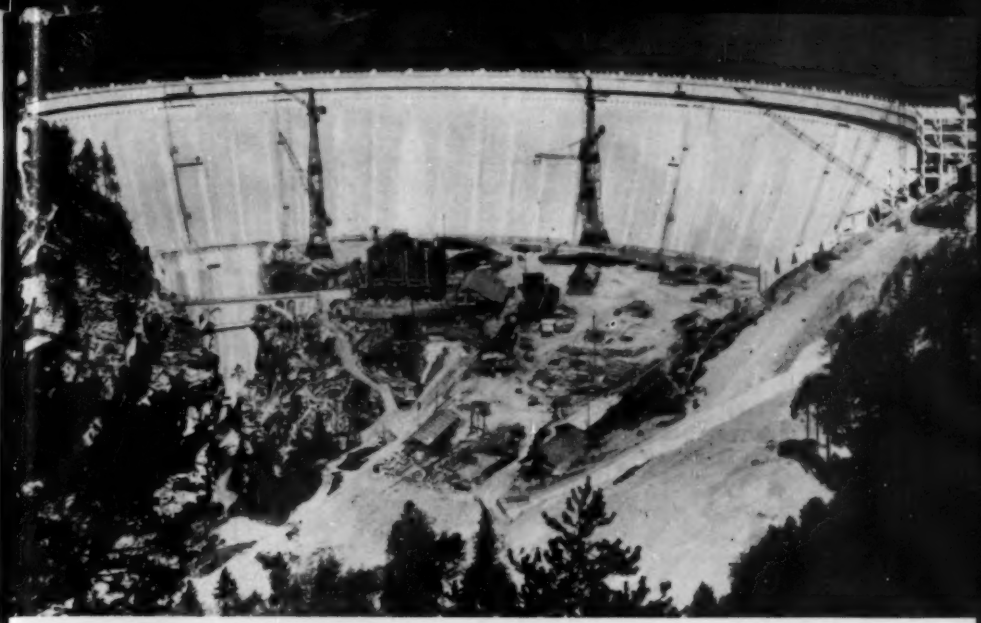

Italy Dam of Pieve di Cadore, Province of Belluno 1946-1948

Arch gravity dam Max. height: 112 meters Length of crest: 410 meters Span: 308 meters

Concrete—494,000 cubic yards. A ferro-pozzolanic cement was used consisting of 75% portland cement and 25% pozzolana. The core concrete contained 338 pounds of cement per cubic yard. The face concrete on the water side contained 424 pounds of cement per cubic yard. Plastiment was added in the proportion of 1% by weight of cement throughout.

OTHER DAM PROJECTS

Venezuela Dam on River Caroni 1986-1989

Gravity type dam Height: 30 meters Length: 180 meters

Concrete — 262,000 cubic yards with 4 fluid ounces of Plastiment liquid per bag of cement.

Austria Limberg Dam—Kaprun Project 1948-1951

Arch dam Height: 120 meters Length of crest 350 meters Breadth of crest: 6 meters Breadth of toe: 40 meters

Concrete — 590,000 cubic yards. Mix contained 440 pounds of cement per cubic yard and 1 pound of Plastiment powder per bag of cement. Twenty-eight day compressive strength averaged 4250 psi.

Italy Dam in Val Gallina 1949-1951

Arch dam Max. height: 92 meters Length of crest: 228 meters Span: 190 meters

Concrete — Mix contained 280 pounds of cement and 140 pounds of fly ash per cubic yard. Plastiment was used to assure good workability and specified compressive strength.

Austria Dam of Salza 1947-1948

Arch dam Height: 50 meters Length of crest: 120 meters

Concrete — 36,600 cubic yards. Mix contained 458 pounds of cement per cubic yard and Plastiment. Plastiment was used throughout the later stages of construction to assure reaching specified compressive strengths.

Switzerland Dam of Lucendro, St. Gothard 1943-1947

Buttress dam Height: 70 meters Length of crest: 270 meters

Concrete — 200,000 cubic yards. Face concrete on water side and crest concrete contained 458 pounds of cement per cubic yard and Plastiment powder 1% by weight of cement.

Switzerland Dam of Sella, St. Gothard 1943-1947

Gravity dam Height: 36 meters Length of crest: 300 meters

Concrete — 98,000 cubic yards. Face concrete on the water side contained 458 pounds of cement per cubic yard and 1% Plastiment powder by weight of cement.

Algeria Dam of Beni-Bahdel 1936-1937

Multiple arched dam Height: Max. 57 meters Length of crest over arch: 220 meters

Concrete — Mix contained 508 pounds of cement per cubic yard and 1% Plastiment powder by weight of cement. Plastiment was added to improve strength and workability.

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Mass Concrete – Plastiment Concrete Densifier

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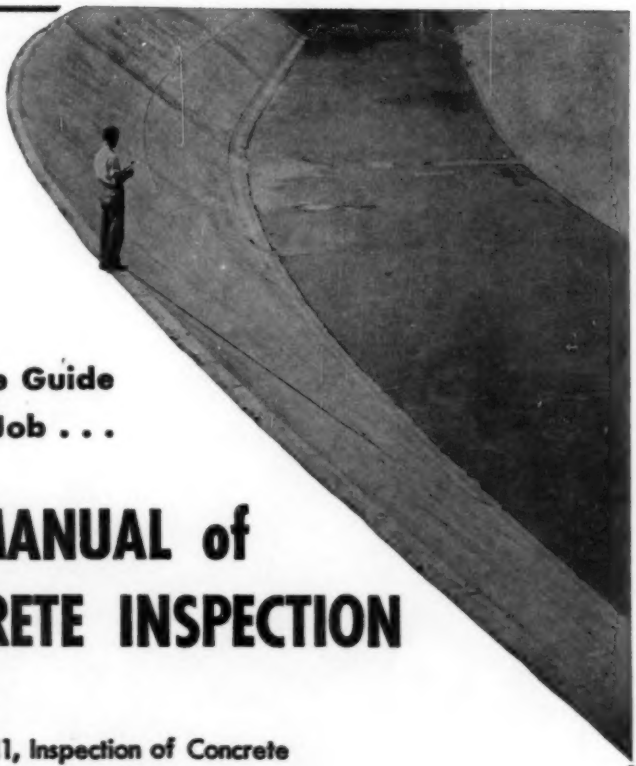

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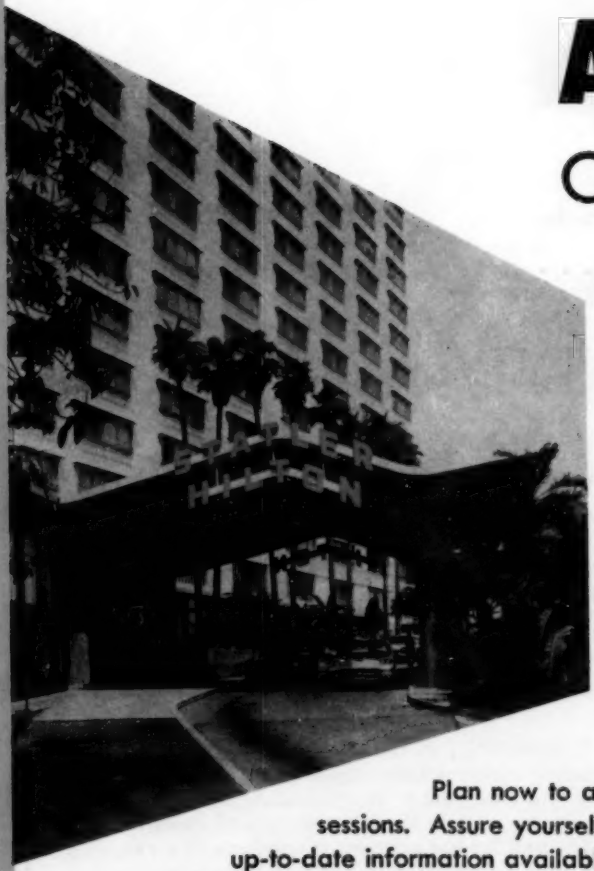

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